53D CONGRESS, 3d Session. } SENATE. { Ex. Doc. No. 12.

IN THE SENATE OF THE UNITED STATES.

LETTER

FROM

THE SECRETARY OF WAR,

IN RESPONSE TO

Senate resolution of December 10, 1894, transmitting the report of the Board of Engineers and bridge-building experts, with other information, relative to the subject of a bridge across the Hudson River at New York City.

DECEMBER 17, 1894.—Referred to the Committee on Commerce and ordered to be printed.

WAR DEPARTMENT,
Washington, D. C., December 14, 1894.

SIR: I have the honor to acknowledge the receipt of the following resolution of the United States Senate, dated December 10, 1894:

Resolved, That the Secretary of War be, and he is hereby, requested to transmit to the Senate the report of the Board of Engineers and bridge-building experts appointed under the act of Congress entitled "An Act to authorize the New York and New Jersey Bridge Companies to construct and maintain a bridge across the Hudson River, between New York City and New Jersey," approved June 7, 1894; and also the report of any board of engineers which may have been appointed by the Secretary of War within the past five years to investigate the subject of a bridge across the Hudson River at New York City; and also to inform the Senate what, if any, action has been taken on either of said reports.

In reply there is transmitted herewith a printed copy of the report of the Board of Engineers appointed by the President, dated August 23, 1894, on page 53 of which will be found attached a copy of the indorsement of the Secretary of War, dated December 12, 1894, approving the report.

With reference to so much of the resolution as calls for "the report of any board of engineers which may have been appointed by the Secretary of War within the past five years to investigate the subject of a bridge across the Hudson River at New York City," there is transmitted an advance copy of a report of a Board of Engineer Officers convened by an order of the Secretary of War, dated January 27, 1894, to "investigate and report their conclusions as to the maximum length of span practicable for suspension bridges, and consistent with an amount of traffic probably sufficient to warrant the expense of construction." This report is now in the hands of the Public Printer, and the appendices are not yet printed.

In connection with the subject there is forwarded a copy of a joint letter from the president of the New York and New Jersey Bridge Company of New York and the president of the New York and New Jersey Bridge Company of New Jersey, dated November 21, 1894. A copy of the reply of the Secretary of War to this communication, dated December 12, 1894, is also herewith.

Very respectfully,

DANIEL S. LAMONT,
Secretary of War.

The PRESIDENT OF THE UNITED STATES SENATE.

[The New York and New Jersey Bridge Company, office of the president, 214 Broadway. New York and New Jersey Bridge Company, 80 Broadway, City.]

NEW YORK, *November 21, 1894.*

DEAR SIR: The New York and New Jersey Bridge Companies, after a careful consideration of the conditions involved in the construction of a bridge crossing the North River between Fifty-ninth and Sixty-ninth streets, have become convinced that, in order to perfect a sound financial basis for the enterprise, it is essential that a guaranteed estimate of cost should be obtained from parties whose reputation and experience in the construction of large works would insure the accuracy of their estimates and the practicability of the plans selected by them.

In accordance with this view, the New York and New Jersey Bridge Companies have entered into a contract with the Union Bridge Company, whereby the Union Bridge Company are to furnish plans and construct a cantilever railroad bridge across the Hudson River, having a main span not exceeding 2,000 feet, at a total cost guaranteed by the Union Bridge Company not to exceed $22,000,000, including interest charges growing due during construction, and to render such assistance as may be in their power in securing the necessary capital for the completion of the bridge.

This contract is of course expressed to be conditional upon the approval by the honorable Secretary of War of the plan for a cantilever bridge having a main span not exceeding 2,000 feet.

Under this contract the Union Bridge Company has presented general plans and estimates for a "cantilever" bridge, having a span of 2,000 feet in the clear, and is prepared to execute the work at a price which it is believed will prove remunerative to the capital invested.

These plans and estimates, together with the reports accompanying the same, have been accepted and adopted by the New York and New Jersey Bridge Companies, and with your kind permission will be presented for your consideration.

The New York and New Jersey Bridge Companies are satisfied that a "suspension" bridge spanning the North River without a pier would involve such elements of uncertainty as regards first cost, novelty in its magnitude as a hitherto untried engineering feat and time of construction, to say nothing of the well-founded prejudice against the "suspension" principle for railroad purposes, as would render the enterprise impracticable from a financial standpoint.

On the other hand, we firmly believe that the "cantilever" plan presented by the Union Bridge Company can be executed within practicable limits of time and cost, and that when finished the bridge will be capable of fully meeting the demands of the heaviest railroad traffic.

In addition to the records contained in the printed report of the Board constituted under the act, a further report of Mr. Macdonald, approved by us, is herewith presented, giving in detail some of our reasons for considering the "suspension" bridge impracticable.

In view of the foregoing facts, we respectfully request that the cantilever bridge, with a main span not exceeding two thousand feet, upon the plans prepared by the Union Bridge Company, may receive your favorable consideration and decision.

Respectfully submitted.

JOHN B. KERR,
Presdt. of New York and New Jersey Bridge Co. of New York.

H. M. HAAR,
Presdt. of New York and New Jersey Bridge Co. of New Jersey.

The SECRETARY OF WAR,
Washington, D. C.

WAR DEPARTMENT,
Washington, December 12, 1894.

GENTLEMEN: I beg to acknowledge receipt of your letter of the 21st ultimo, by which it appears that your company has entered into a provisional contract with the Union Bridge Company for the construction of a cantilever bridge between New York and New Jersey on plans involving the erection of a pier within the lines of the navigable waters of the harbor of New York.

The present Congress, at its first session, passed an act which would have permitted the construction of a bridge such as you now propose. After much consideration and a public hearing, where representations of the commercial organizations speaking for the vast interests of the commerce of that port affecting all sections of the country were submitted and the public demands for quick and convenient transit over the river at this point were expressed, that bill failed to receive Executive sanction, and was returned to the House of Representatives where it originated.

The reasons for disapproval were stated in part in the veto message, as follows:

This bill authorizes the construction of a bridge over the North River between the States of New York and New Jersey, the terminus of which in the city of New York shall not be below Sixty-sixth street. It contemplates the construction of a bridge upon piers placed in the river; no mention is made of a single span crossing the entire river, nor is there anything in the bill indicating that it was within the intention of the Congress that there should be a bridge built without piers. I am by no means certain that the Secretary of War, who is invested by the terms of the bill with considerable discretion so far as the plans for the structure are concerned, would have the right to exact of the promoters of this enterprise the erection of a bridge spanning the entire river.

Much objection has been made to the location of any piers in the river for the reason that they would seriously interfere with the commerce which seeks the port of New York through that channel. It is certainly very questionable whether piers should be permitted at all in the North River at the point designated for the location of this bridge. It seems absolutely certain that within a few years a great volume of shipping will extend to that location which will be seriously embarrassed by such obstructions.

I appreciate fully the importance of securing some means by which railroad traffic can cross the river, and no one can fail to realize the serious inconvenience to travel caused by lack of facilities of that character. At the same time it is a plain dictate of wisdom and expediency that the commerce of this river be not unnecessarily interfered with by bridges, or in any other manner.

Engineers whose judgment upon the matter can not be questioned, including the engineer of the company proposing to build this bridge, have expressed the opinion that the entire river can be spanned safely and effectively by a suspension bridge, or a construction not needing the use of piers.

The company to which the permission to bridge the river is granted in the bill under consideration was created by virtue of an act of the legislature of the State of New York, which became a law, by reason of the failure of the governor to either approve or veto the same, on the 30th day of April, 1890. It may be safely assumed that the members of the legislature which passed this law knew what was necessary for the protection of the commerce of the city of New York, and had informed themselves concerning the plan of a bridge that should be built in view of all the interests concerned. By paragraph 24 of the law creating this company it is provided that "the said bridge shall be constructed with a single span over the entire river between towers or piers located between the span and the existing pier-head lines in either State," and "that no pier or tower or other obstruction of a permanent character shall be placed or built in the river between said towers or piers under this act."

In view of such professional judgment, and considering the interests which would be interfered with by the location of piers in the river, and having due regard to the judgment of the legislature of the State of New York, it seems to me that a plan necessitating the use of piers in the bed of the river should be avoided. The question of increased expense of construction or the compromise of conflicting interests should not outweigh the other important considerations involved.

This message was referred to the Committee on Interstate and Foreign Commerce, and subsequently a bill was introduced "intended to conform to and meet the objections urged by the President." In presenting this substitute bill to the favorable consideration of Congress that committee submitted a report which I am justified in assuming clearly shows the purpose of Congress in its subsequent legislation upon this subject, and from which I make the following extracts:

The main objection urged by the President to the bill heretofore passed was that it allowed a bridge to be built with a pier in the river. * * *

The bill as now presented directs the President to create a board of five competent, practical, disinterested, expert bridge engineers, of whom one shall be a member of the Corps of Engineers, United States Army, and the others from civil life, who shall, in the time stated, meet, investigate and examine into, determine and decide, at what length of span not less than two thousand feet a safe, practicable railroad bridge can be constructed, and make their determination as to the length of the span final. As this is the chief difference between the executive and legislative departments in the passage of a bill to bridge the Hudson River, which is not only a question of State but of national importance, there can be no better plan devised or determined, at what span the river can be safely bridged, than by a board of the character described in the act and appointed under the authority of the President.

Bridge-building is a science which has advanced probably as much as any other within the last twenty years. What was impracticable twenty years ago is practicable now; what could not be safely done twenty years ago may be safely done now, and there is no more accurate way of ascertaining what can be scientifically done, from a practical standpoint, than the one proposed in the bill for ascertaining what length of span can be made, taking into consideration all the possibilities of the weight and strain on the steel of which it is to be constructed. It is far safer, in the opinion of the committee, to leave this important question to disinterested expert bridge-builders to be selected by the President than even to the opinion of the engineers of the War Department, however able they may be in other branches or other lines of engineering. These questions will be familiar to the commission. They have dealt with many such problems, having in mind not only the possibility of length of span, but also the obstruction which may occur to the navigation of the river. And their opinion is certainly, when disinterestedly expressed, the most valuable verdict which can be finally rendered in settlement of a dispute in a matter of this character.

It will be seen that * * * all plans for the construction of the bridge as to length of span must conform to the decision of this board.

* * * * * * *

The committee are satisfied that if, in the opinion of the board of competent engineers established under the act, a safe and practicable bridge can be built of a greater length than 2,000 feet, as shown in the plans of the bridge companies exhibited to the committee, they will, as they must, adopt it.

They do not believe that any better or more reliable way can be found than to leave it to a settlement by a board of competent engineers, selected by the President himself, and sitting under the direction of the Secretary of War.

The act, as thus presented, was passed by both Houses, and was approved by the President June 7, 1894. Under its provisions the President appointed Maj. Charles W. Raymond, of the Corps of Engineers, United States Army; Prof. William H. Burr, of Columbia College; G. Bouscaren, of Cincinnati; George S. Morison, of Chicago; and Theodore Cooper, of New York, "as a board of competent, disinterested engineers to examine and recommend what length of span not less than two thousand feet would be safe and practicable for a railroad bridge to be constructed over said river." After a thorough investigation covering every phase of the problem this Commission submitted an exhaustive report, signed by all of its members, which closed with the following recommendation:

The only subject referred to your Board is "to recommend what length of span not less than 2,000 feet would be safe and practicable for a railroad bridge to be constructed over the Hudson River between Fifty-ninth and Sixty-ninth streets." A single span from pier-head to pier-head, built on either the cantilever or suspension principle, would be safe. The estimated cost of the 3,100-foot clear-span cantilever being about twice that of the shorter span, your Board consider themselves justified in pronouncing it impracticable on financial grounds. As the cost of the single-span suspension bridge is at most (not more than) one-third greater than that of the 2,000 cantilever, your Board are unable to say that such greater cost is enough to render the suspension bridge impracticable.

The Board have reached this conclusion after careful study, and they have thought it best to give the full course of reasoning which they have followed. They feel that the contingency attending the construction of the deep-river foundation of the cantilever bridge, even waiving the absolute necessity of carrying this foundation to rock, is enough to balance a part of the greater cost of the suspension bridge.

The conclusion of this Board is that of a Board of Bridge Engineers acting in a professional capacity. While from such professional view they must pronounce the suspension bridge practicable, they do not in this conclusion give an opinion on the financial practicability and merit of either plan.

This finding is confirmed and strengthened by the unanimous report of a Board of Officers of the Corps of Engineers of the United States Army, appointed prior to the legislative provision for a Board, with instructions to "investigate and report their conclusions as to the maximum length of span practicable for suspension bridges and consistent with an amount of traffic probably sufficient to warrant the expense of construction."

The conclusion of this Board, which is indorsed by the Chief of Engineers of the United States Army, has been reported to me as follows:

The final plans for a work of such magnitude would only be adopted after the most extended theoretical and experimental investigations, and the estimated cost would undoubtedly be much reduced by such studies. Assuming the most favorable location and the most competent engineering management, the Board believe that $23,000,000 is a reasonable estimate for a six-track railroad suspension bridge 3,200 feet long, and they consider the amount of traffic which such a bridge would accommodate sufficient to warrant the expense of construction. They believe, however, that the bridge should be so constructed that its capacity can be readily increased, and with the suspension system this can be provided for by giving suitable dimensions to the towers and anchorages.

If sufficient inducements were offered to competent engineers to prepare competitive designs and estimates for a single-span bridge at this locality, the Board do not doubt that perfectly satisfactory plans would be obtained within the limit of cost of the estimate given above.

Briefly stated, the finding of the engineers is that both the 2,000-foot cantilever bridge which requires a pier in the river, and the 3,100-foot in the clear suspension bridge without a center pier are safe and not impracticable as to cost, the estimates being as follows:

PIER BRIDGE (CANTILEVER) SIX TRACKS.

[Clear span, 2,000 feet.]

Total length, 4,320 feet; moving load, 3,000 pounds per foot of track...	$25, 443, 000
Total length, including necessary viaduct, making it equal in length to a suspension bridge having a clear span of 3,200 feet, 5,600 feet; moving load, 3,000 pounds per foot of track	26, 723, 000

SUSPENSION BRIDGE, SIX TRACKS.

[Clear span, 3,200 feet; total length, 5,600 feet.]

Moving load, 3,000 pounds per foot of track	$35, 367, 671
Moving load, 1,500 pounds per foot of track; strain on cables, 60,000 pounds per square inch	30, 743, 000
Moving load, 1,500 pounds per foot of track; strain on cables, 50,000 pounds per square inch	31, 671, 000
Location near Sixty-ninth street, moving load, 3,000 pounds per foot of track	31, 917, 671
Location near Sixty-ninth street, moving load, 1,500 pounds per foot of track	27, 771, 000
Location near Sixty-ninth street (Army Engineer Board), moving load, 1,500 pounds per foot of track	23, 000, 000

And I am further assured by members of the Commission that the time required for construction would be approximately the same.

In view, therefore, of these facts and findings, and of the very serious objections to a pier in the harbor, with its attendant delay and constant menace to navigation, I am constrained to require that the bridge to be built by the New York and New Jersey Bridge Companies across the Hudson River shall have a single span between the pier lines of the harbor of New York.

While I do not undertake to say that the objection of an obstruction in the harbor is sufficiently grave to prohibit the construction of a pier bridge, were a suspension bridge impracticable, it is a matter for public congratulation that a scientific investigation of the subject by skilled and experienced engineers determines that both the traffic on the river as well as that over it can be accommodated without interference and the rights of all protected with only such an increase of expense, if any, as is clearly demanded by the conceded advantages to follow.

Very respectfully,

DANIEL S. LAMONT,
Secretary of War.

Mr. JOHN B. KERR,
Presdt. of New York and New Jersey Bridge Co. of New York.
and
Mr. H. M. HAAR,
Presdt. of New York and New Jersey Bridge Co. of New Jersey.

OFFICE OF THE CHIEF OF ENGINEERS,
UNITED STATES ARMY,
Washington, D. C., October 25, 1894.

SIR: I have the honor to submit herewith a printed copy of the report of the Board of Engineer Officers convened in accordance with your order, dated January 27, 1894, to investigate and report their conclusions as to the maximum length of span practicable for suspension bridges. The report is a very valuable one, shows careful research, and I approve and concur in its conclusions.

Very respectfully, your obedient servant,

THOS. LINCOLN CASEY,
Brig. Gen., Chief of Engineers.

Hon. DANIEL S. LAMONT,
Secretary of War.

REPORT

OF

BOARD OF ENGINEER OFFICERS TO MAKE INVESTIGATIONS OF CERTAIN BRIDGES.

UNITED STATES ENGINEER OFFICE,
Philadelphia, Pa., September 29, 1894.

GENERAL: The Board of Officers of the Corps of Engineers appointed by Special Orders No. 5, current series, Headquarters Corps of Engineers, U. S. Army, January 29, 1894, to make investigations as to certain bridges, in accordance with instructions of the Secretary of War, have the honor to submit the following report:

The instructions of the Board are contained in a letter from the Secretary of War to the Chief of Engineers, dated January 27, 1894, and in the indorsement of the Chief of Engineers thereon, dated January 30, 1894. Copies of this letter and the orders convening the Board are appended hereto.

The Secretary of War, in his letter of January 27, 1894, remarks that, "in view of the importance of questions arising in this Department in connection with the building of bridges over navigable streams, it is essential that it should be possessed of accurate and full information necessary to their intelligent and proper determination;" and directs the formation of "a Board of Officers of the Engineer Corps, who shall investigate and report their conclusions as to the maximum length of span practicable for suspension bridges and consistent with an amount of traffic probably sufficient to warrant the expense of construction."

The indorsement of the Chief of Engineers of January 30, 1894, directs the Board to include in its investigations "strength of materials, loads, foundations, wind pressure, oscillations, and bracing."

The Board convened at New York City on February 13, 1894, and remained in session until February 15, 1894, when it adjourned to collect information. The Board held a session at Philadelphia, Pa., from March 6 to 10, 1894. An extended preliminary investigation of the subject under consideration had already been made when a Board of expert bridge engineers was appointed by the President, on June 15, 1894, under the provisions of the act approved June 7, 1894, to recommend what length of span, not less than 2,000 feet, would be safe and practicable for a railroad bridge to be constructed across the Hudson River between New York City and the State of New Jersey. The New York Board was composed of five engineers, four of whom were civil engineers of long and varied experience in the designing and construction of bridges and of the highest profes-

sional standing. It was therefore considered desirable to delay the completion of this report until the determinations of the New York Board could be ascertained and studied. The present Board has derived much assistance from the published report of the New York Board, as will be indicated below. At the last session, which was held at Philadelphia from September 20 to September 29, 1894, this report was unanimously adopted.

The question of the maximum practicable span may be investigated as a purely engineering problem, when certain preliminary conditions are established. The bridge will doubtless be a railroad bridge, since with the largest span the traffic capacity would not otherwise justify the cost. This assumed, the width must be at least sufficient to accommodate a double track. The number of double tracks required must be established so as to give a traffic capacity "probably sufficient to warrant the expense of construction."

The New York and Brooklyn bridge, the longest suspension bridge yet constructed, consists in reality of two similar bridges suspended side by side and braced together, the promenade being supported between the bridges as an extra weight on the interior cables. Following this idea, the Board in its preliminary investigation assumed a double-track railroad bridge as the unit bridge, bracing together side by side as many such bridges as were considered necessary to accommodate the traffic contemplated. The engineering problem was thus limited to the question of determining the maximum span for a double-track railroad bridge. It was found, however, that there are many serious practical objections to such an arrangement in a long-span bridge carrying very heavy loads. In this investigation, therefore, the loads will be assumed to be supported by only two sets of cables, one on each side of the bridge; an arrangement which was adopted as a basis of estimate by the New York Board.

In the various projects for long-span bridges across the Hudson River at New York the least traffic capacity assumed was six tracks, and the New York Board adopted this number of tracks in its investigations. In this report, therefore, it is proposed to first consider the question of the maximum span for a six-track railway bridge as an engineering problem, after which the relations between span, traffic, and cost of construction will receive such investigation as the nature of the subject will permit.

Since much of the information with reference to strength of materials, loads, etc., collected by the Board as directed by the Chief of Engineers, is necessary for the proper investigation of the question of the practical maximum span, this part of the subject will first receive attention.

STRENGTH OF MATERIALS.

The supporting cables of a suspension bridge of long span are made of steel. They are either chains composed of connected links or cables formed of parallel wires or twisted wire ropes. To obtain the longest span possible the weight of the cable must be a minimum as compared with its carrying capacity. The connections of a series of links add from 20 to 25 per cent to the dead weight of the chain, while in the wire cable the connections add, at most, only 2 or 3 per cent. Moreover, steel in the form of wire has a minimum strength more than double its maximum strength in the form of bars suitable for the construction of a suspension chain. A link chain, therefore, will weigh about two

and one-half times as much as a wire cable of equal carrying capacity; or in other words, a wire cable can be stretched about two and one-half times as far as a steel chain before being broken, other conditions being the same in both cases. Moreover, it is stated by Melan* that considerable bending moments are sometimes produced by the friction between the links of chains, but the effects due to the stiffness of cables are so small that they may be neglected. It is therefore assumed that the cables are made of steel wires laid parallel to each other.

The strength of the suspension cable will depend upon the tensile strength of the steel employed in its construction and upon the number of wires it contains. The wire employed in the cables of the New York and Brooklyn bridge had a tensile strength of 170,000 pounds per square inch, and the cables were originally designed to contain each 6,188 wires of No. 7, B. W. G., but as some heavier wires were introduced during construction, the actual number of wires was only 5,400. These are the largest cables made up to the present time, having a diameter of 15⅝ inches. The cables of the Cincinnati suspension bridge have a diameter of 12 inches and each contains 5,200 No. 9 wires.

There is a practical limit to the number of wires which can be united in a cable, since as the number increases it becomes more and more difficult to adjust the wires so that each will bear its due proportion of stress under the varying conditions of temperature and loading. No unusual difficulties, however, were encountered in the manufacture of the cables above referred to, but it is believed that with the method employed for making the cables of the East River bridge the practical limit of the number of wires was very nearly, if not quite, attained. With improved methods the construction of much larger cables might be found practicable. An increase in the size of the wire does not materially increase the difficulty of construction. No. 3 wire having a tensile strength of 180,000 pounds per square inch, can now be readily obtained at a reasonable price. Indeed steel wire much stronger than this can be obtained (up to more than 300,000 pounds per square inch) but its present cost would prohibit its employment.

The Board therefore assumes for the purposes of this investigation a suspension cable formed of 6,000 parallel steel wires, No. 3, B. W. G. The area of its cross-section will be 316 square inches without wrapping, and its breaking tensile strength will be 56,880,000 pounds, or 28,440 tons. With a safety factor of 3, which was adopted by the New York Board, and will be adopted in this investigation for reasons to be given hereafter, the working strength of this cable will be 18,960,000 pounds or 9,480 tons. Its diameter with wrapping will be 21½ inches. The New York Board have adopted a cable of about this size and strength in their estimates.

The total cable strength available for the support of the bridge depends upon the number of cables which can be practically combined as a single cable system on one side of the bridge. If many cables are employed it becomes difficult to distribute the strains among them so that each shall carry its proportionate load under the varying conditions of temperature and traffic. It is not easy to decide what is the practical limit of the number of cables to be assembled together. Where parallel wire cables are used they must be sufficiently separated horizontally and vertically to give room for the operation of the wire-

*Handbuch der Ingenieurwissenschaften. Band II. Der Brückenbau—J. Melan, Leipzig, 1888.

wrapping machine and this requires intervals of at least 3 feet during construction.

The cables may cross the saddles on top of the towers side by side or they may be arranged in one or more vertical or nearly vertical planes. In the first case the cradling of the cables in converging planes (which is desirable for lateral stability), requires considerable intervals between the saddles. In the investigations of the New York Board an interval of 20 feet was found to be necessary for such an arrangement. But the cables on one side of the bridge may also be arranged side by side in parallel inclined planes and held the same distance apart throughout their length by iron separators between the suspender clamps, in which case the saddles on the towers would be closer together. Still the number of cables suspended on each side can not be made very large without increasing the dimensions of the towers and the piers supporting them far beyond the requirements of the roadway and suspended loads.

The vertical arrangement of the cables (or a combination of vertical and horizontal arrangements) certainly presents some very decided advantages. It requires less width at the top of the towers, and a large part of the stiffening of the bridge may be obtained by trussing the cables in a manner which will be again referred to. Moreover, with this arrangement the towers can be so constructed that new cables can be readily added to meet future demands for increased traffic capacity. This method, however, is not so simple as the other, and with large loads and cables involves mechanical difficulties which can be properly dealt with only after an extended investigation of the problem as a special case. In this general investigation the Board consider it best to adopt the simpler arrangement, as has been done by the New York Board. Whatever arrangement is adopted, the Board are of the opinion that it would not be found convenient to work more than eight cables together as one cable system. For the purposes of this investigation, it is therefore assumed that the suspension bridge of maximum span is supported by sixteen 21½-inch cables. The following list giving the arrangements employed in a number of important bridges, may be of interest in this connection:

New York and Brooklyn Bridge, 1,595.5 feet span, has *one* 15⅝-inch cable on each side.

Niagara Bridge, 821.3 feet span, *two* 10-inch cables, one vertically over the other.

Wheeling Bridge, 1,010 feet span, *two* 8-inch cables, side by side.

Fairmount Bridge, 550 feet span, *seven* cables, 6 side by side and 1 above.

Freiberg Bridge, Switzerland, 870 feet span, *three* cables, 2 side by side and 1 above.

Dordogne Bridge, France, 350 feet span, *three* cables, side by side.

Niagara Bridge, at Lewistown, 1,400 feet span, had *four* cables side by side.

Menai Bridge, 600 feet span, *four* chains in the same vertical plane.

Tweed Bridge, Berwick, 450 feet span, *three* chains in the same vertical plane.

Tersing Bridge, over the Maas, *two* chains, one over the other.

Bridge Voconflans, St. Honorine, *two* chains, one over the other.

La Roche Bernard Bridge, over the Vilaine, 650 feet span, *two* cables, side by side.

Lambeth Bridge, England, *two* cables, side by side.

Donau Bridge, Pesth, 660 feet span, *two* chains, one over the other.

Moldau Bridge, Prague, *four* chains in pairs, over each other.

LOADS.

The total load supported by the bridge will consist of two parts, viz, the live load or weight of the passing traffic and the dead load or weight of the structure. It will be convenient to consider the load under these two heads.

1. *Live load.*—In another part of this report the difference in the character of the action of the live load upon a structure in stable equilibrium having a certain degree of flexibility from that of its action upon a more rigid structure in unstable equilibrium will receive due consideration. For the present we will only determine the magnitude of the live load. The live load per linear foot of span will be represented by q.

The greatest static effect upon the cable will be produced by the maximum load; that is, when the whole platform from tower to tower is covered with the heaviest possible railroad trains.

A suspension railroad bridge of very long span will as a rule be built only over a wide river or estuary navigable by ocean craft and therefore requiring a great height of the bridge above the water. To limit the expense of the shore extensions the approaches must be given as steep a grade as is admissible for a railroad bridge. It is therefore assumed that the approaches will have a grade of 1 per cent.

The weight of a railroad train passing over the bridge need not be considered as any greater than that which the heaviest freight locomotive is capable of hauling up a 1 per cent grade. From a list published by the Baldwin Locomotive Works it appears that exceptionally heavy locomotives are built with 170,000 pounds on the drivers and a total weight of 192,500 pounds. The Baldwin Works allow 9 tons for each 1,000 pounds on the drivers as the maximum efficiency on a grade of 1 per cent. This extra heavy locomotive can therefore pull up on the bridge 1,530 tons including its own weight. Subtracting 96 tons for the weight of the locomotive, we have for the weight of the train, 1,434 tons. This is equal to 41 hopper-bottom gondola cars each 27 feet 2 inches long and weighing 35 tons. The length of the engine and tender being 54 feet, the total length of the train will be 1,168 feet, and the weight per linear foot of track will be 2,620 pounds, equal to 1.31 tons. If we suppose all 6 tracks to be loaded from end to end with such trains, the live load per linear foot will be 15,720 pounds, equal to 7.86 tons.

Any such loading as this, however, is so extremely improbable as to be a practical impossibility; indeed, at the height above water level at which such a bridge must be carried, the transportation of passengers, and not of freight, must be the main consideration. A purely freight traffic would in no conceivable location require six tracks over a bridge of very long span. Such a number of tracks would only be justified by the location of the bridge near a very large city and by a large passenger traffic. The trains passing over such a bridge would undoubtedly be controlled on the block system and not more than one train on each track would be allowed upon the bridge at the same time. If the stiffening girders could do their full duty the weights upon the bridge would be uniformly distributed and the live load per linear foot would be $\frac{3,060,000}{L} \times 6 = \frac{18,360,000}{L}$ pounds. The distribution, however, is not perfectly uniform, and there are occasionally other causes which produce an increase in local stresses. The Board consider it best to add 50 per cent to this estimate to cover these uncertainties. In

computing the cable strength, therefore, the adopted value of the live load will be—

$$q = \frac{27{,}540{,}000}{L}$$

The live load assumed for the Niagara Suspension Bridge, which has a span of 800 feet, is 350 tons in a length of 450 feet of single track, which is equivalent to 1,600 pounds per linear foot of train, or less than 1,000 pounds per linear foot of the entire span, with a factor of safety of 4.41 in the cables and 4.0 in the stiffening girders.* The bridge, however, safely carries heavier trains in daily operation.

The largest existing railroad bridge is the Forth bridge, in Scotland. It has 2 tracks and 2 spans of 1,700 feet each. It was tested with 2 heavy trains side by side, each 1,000 feet long and weighing 900 tons. Each train was drawn by 2 locomotives each weighing 72 tons. This load was equivalent to 1,800 pounds per linear foot of track, or 3,600 pounds per linear foot of span. These were considered extra heavy train loads, very seldom occurring in actual operation on English roads.†

Making allowance for the heavier train loads of American railroads, it will be seen that 3,000 pounds per linear foot of track for a length of 1,500 feet, considered as distributed over the bridge from tower to tower (which is the value given by our formula), is an exceedingly safe assumption of the live load for a very long bridge in which the span exceeds the length of the maximum train. This value agrees very nearly with that assumed by the New York Board in its estimate for a "lighter structure."

It is very evident that the assumed live load per unit of track ought to diminish with the number of tracks and with the length of span. A single-track bridge of short span is strained nearly to its maximum every time a train goes over it. A 6-track bridge is strained to its maximum only when 6 maximum trains are abreast of each other; and when the span exceeds the maximum train length the maximum stress ought never to occur.

2. *Dead load.*—This produces at all times constant strains in the members of the bridge. It is composed of the weights of the following parts: The suspension cables with their wrapping, the platform, the stiffening girders, the wind and sway bracing, and the suspenders. The weights of these parts per linear foot of span will be represented as follows: Cables $= w$; cable wrapping $= w_0$; platform $= p_1$; girders $= p_2$; bracing $= p_3$; suspenders $= p_4$.

(1) *Weight of the suspension cables.*—The weight of a cable formed of 6,000 parallel steel wires, No. 3, B. W. G., having a diameter of $21\frac{1}{4}$ inches, without wrapping, will be 1,075 pounds, or 0.538 tons per running foot. If we assume 16 cables for the support of the entire bridge, the total cable weight per linear foot will be $w' = 17{,}200$ pounds $=$ 8.6 tons; and $w = 17{,}917 = 8.959$ tons.

(2) *Cable wrapping.*—The cable will be wrapped with iron wire of No. 9, B. W. G. The weight of this wrapping will be 26 pounds per linear foot, and for 16 cables, 416 pounds. The weight of wrapping per linear foot of span will therefore be $w_0 = 433$ pounds.

It will be found convenient for the purposes of this investigation to know the relation between the weight of the cable per linear foot of horizontal span (w) and its weight per foot measured in the direction

*Proceedings Am. Soc. C. E., Vol. x, p. 195.

†Record of the Forth Bridge, p. 64.

of its axis (w'). The cable curve will be approximately a parabola, and its approximate length will be $\left(1+\frac{2.67}{R^2}\right)$ L, in which L is the horizontal span in feet and R is the ratio of the versine of the cable to the span. Hence $w=\left(1+\frac{2.67}{R^2}\right)w'$. For reasons which will be given in the sequel the value of R will be assumed as 8 in this investigation. For this value we have $w=1.0417\ w'$ and $w'=0.960\ w$.

(3) *Weight of the platform.*—In the arrangement adopted in this investigation for the purposes of estimate the cross-girders sustain the weight not only of the live load but also of the stiffening girders and lateral bracing. The distance between the cross-girders is 30 feet.

The platform further consists of longitudinal girders (stringers), the permanent way consisting of ties and rails and the covering. Its construction is the same as in any other railroad bridge.

The weights of the stiffening girders and lateral bracing (which increase with the span), are imposed upon the cross-girders so very near the points of suspension that the weight of the platform may be considered practically independent of the span. The weight per linear foot of span for a 6-track platform, carrying stiffening girders and bracing, for a span of 3,200 feet, was determined by the New York Board to be 7,200 pounds. We may therefore adopt in this investigation the constant value $p_1=7{,}200$ pounds$=3.6$ tons.

(4) *Weight of the stiffening girders.*—It will be shown under the head of Vertical Bracing that the weight of the two stiffening girders per linear foot of 6-track bridge may be found in pounds from the formula $p_2=3{,}281+2.754\ L+0.0005312\ L^2$. This includes an allowance of material in the lower chords to provide for the stresses due to wind.

(5) *Weight of the lateral bracing.*—As will be shown under the head of Lateral Bracing, the weight of the wind and sway bracing per linear foot of 6-track bridge (so far as not provided for in the preceding paragraph) will be given in pounds by the formula $p_3=2{,}420+0.3889\ L$.

(6) *Weight of the suspenders.*—The suspended load is connected with the cables by 8 wire suspenders on each side of the bridge at each cross-girder. The suspenders at each girder are equal in length and are supposed to be adjusted so as to carry practically equal portions of the load. At the middle of the span the cables are 60 feet above the level of the suspension pins. The versine of the cable being $\frac{L}{8}$ the average length of a suspender is $\frac{L}{24}+60$. Assuming a unit working stress of 30,000 pounds and adding 20 per cent for constructive details, the weight of the suspenders per linear foot of span will be

$$p_4=\frac{3.4\times1.2}{30{,}000}\left(\frac{L}{24}+60\right)(p'-w_0)$$

in which $p'=q+p_1+p_2+p_3+p_4+w_0$.

For the purposes of this computation we may assume $p_4=1{,}300$ in the value of $p'-w_0$.

From values previously given we find $p'-w_0=14200+27540000\ L^{-1}+3.1429\ L+0.0005312\ L^2$; and $p_4=272+224726\ L^{-1}+0.10616\ L+0.00002215\ L^2+0.000000003\ L^3$.

(7) *Total suspended weight.*—The total suspended weight per linear foot of span exclusive of the cables will be $p' = q + p_1 + p_2 + p_3 + p_4 + w_0 = 13605 + 27764726\ L^{-1} + 3.24906\ L + 0.00055335\ L^2 + 0.000000003\ L^3$.

WIND PRESSURES.

The wind pressure upon a large bridge is of such magnitude as to require especial consideration. In the principal members of the Forth bridge (a cantilever construction), the maximum stresses due to wind have been stated by its engineer to be more than one-quarter greater than those due to the dead weight of the bridge and nearly three times as great as those due to the live load of passing trains. In suspension bridges these wind stresses, though they may be less than in other bridges, are still of very great importance, and must be carefully provided against by the introduction of metal which, while adding nothing to the carrying capacity of the bridge, does add considerably to its dead load, and therefore necessitates an increase in the strength of suspenders, main cables, towers, anchorages and foundations, and thus may add enormously to the total cost of the bridge. Under such circumstances, while it is, on the one hand, important to secure a sufficiency of wind bracing, it is, on the other hand, equally important not to use any more than is actually necessary.

Since the existing fund of information as to wind pressures, as to their effect on bridges, and as to the present most used methods of dimensioning wind bracing, is either quite limited or else is not easily accessible, it has been thought well to attach hereto a full history of past work in this direction with suggestions of rules for use in the dimensioning of large structures in places exposed to heavy winds. (*See* Appendix C.)

Past history shows the possibility, at almost any place, of an occasional tornado of power sufficient to destroy almost any existing engineering structure. Such tornadoes, like violent earthquakes, are so rare that no large constructions of to-day are made thoroughly proof against them. In such a tornado, however, a suspension bridge would fare much better than any other form of bridge, as it offers the least surface to the wind, as its overturning is almost a physical impossibility, and especially as the loss of large parts of its roadway and stiffening trusses would not necessarily destroy its main cables and towers (these being its essential and costly features). The Board have therefore (for reasons stated in the Appendix) considered a maximum steady wind pressure of 30 pounds per square foot over the entire structure and over a continuous train, reaching entirely across the bridge, and also a similar 30-pound pressure over the unloaded bridge, accompanied by an added pressure of 20 pounds per square foot (making 50 pounds in all) over 1,000 feet of the unloaded bridge; this latter allowance being made to provide for occasional severe gusts.

The exposed surface of the bridge and load per running foot (by the method of calculation given in full in the Appendix) is taken at 30 square feet for the cables and suspenders, 49 square feet for the stiffening girder (including the upper chord, lower chord, web members, horizontal diagonals, and sway bracing), 18 square feet for the platform (including track, guard rails, ties, cross-girders, and stringers), and 8 square feet for the train (excluding the portion sheltered by the high bottom chords and other adjacent parts of the stiffening girders). In view of the heavy weights and consequent great inertia of the cables and stiffening girders, the resulting wind pressure is treated as uniformly distributed over the entire bridge from end to end; though a

more careful distribution, perhaps saving considerable metal, would be adopted in actual practice.

The cradling of the main cables and suspenders is considered sufficient to amply resist the 1,050 pounds per square foot of wind pressure due to the 30 square feet per linear foot of their own surfaces.

The wind bracing of the stiffening truss will then have to resist only the wind pressure on the stiffening truss, platform, and train; which amounts to 2,345 pounds per linear foot for the unloaded bridge, and 2,250 pounds per foot for the loaded bridge.

As the stiffening truss is hinged at its middle, the wind bracing (at least near the ends of the half trusses) must be arranged to carry all the wind stresses to the bottom chord; so that this bracing is taken as composed of a very light upper horizontal truss (not theoretically necessary), of a strong vertical sway bracing, and of a heavier lower horizontal truss. Since the wind trussing may be combined with the adjacent parts of the stiffening girder and platform, this lower wind truss will be built up by increasing, where necessary, the dimensions of the cross girders of the platform, and the lower chords of the stiffening truss, and by inserting cross diagonals between them.

Because of the great size of these cross girders and lower chords, and therefore the great excess of strength in this lower truss when the bridge is unloaded, the Board regard the 2,250 pounds per linear foot of wind pressure on the loaded bridge as the one which throws the greatest strain upon its members, and therefore take this value as the one to be used in combination with the other loads upon a bridge of maximum length.

In case further lateral stiffness against wind should, at any time, be thought desirable, it may be obtained by the use of horizontal wind cables under the platform; and small main cables may at any time be added, if found necessary, to support the added weight of such wind cables; but the Board consider the use of such wind cables unnecessary.

OSCILLATIONS.

In considering the character and importance of small motions in bridges it is necessary to distinguish carefully between stability and rigidity. A suspension bridge is the most stable of all bridge structures. The locus of the centers of gravity of its vertical cross-sections lies far below the points of support. The live load, which is the main cause of its vertical oscillations, always moves below the gravity-line, thereby increasing the lateral stability of the structure. As the span is increased the gravity-line rises, but the resulting slight decrease in stability is more than compensated for by the diminution in the ratio of the live to the permanent load. The small motions of erect-arch and deck bridges must be carefully confined within small limits to insure the safety of the structures, but there is no such necessity in the case of suspension bridges, where the system is in stable equilibrium and sure to return to its position of rest whatever may be the magnitude of the displacement.

The lateral oscillations are due mainly to the action of the wind. These are met not only by the great weight of the structure, but also by the cradling of the cables, which much increases the lateral stability.

It is possible to construct a suspension bridge so that it will have any degree of rigidity desired, but it will appear from the above that rigidity is in this case of much less importance than it is in most other kinds of bridges; indeed, it may be shown that a certain small flexibility is a positive advantage in suspension bridges.

The Board do not consider it necessary to give in this report an elaborate development of the theory of bridge oscillations, because it is perfectly easy to stiffen a suspension bridge so as to reduce both its vertical and lateral deflections, and consequently the duration of its oscillations, within any desired limits; and in bridges of very long span, where the ratio of live to dead load is comparatively small, no difficulty from this cause need be anticipated. The following brief remarks on this subject relate to suspension bridges of comparatively small weight and span.

As before remarked, oscillations in suspension bridges are mainly produced by the impulses of the moving load and by the pressure of the wind. The magnitude of the oscillations is sometimes increased by the lengthening of the central part of the cable, due to the straightening of the chains of the side span under the action of a load on the central span. Theoretically an infinitely small impulsive force may produce an infinitely small amplitude of oscillation of finite duration. If such impulses are repeated a summation of their effects may result. This will occur when the interval between two impulses is equal to the time of oscillation, and may occur when it is greater. Under these circumstances small impulsive forces, by many repetitions, may produce a great oscillation in an elastic or suspended body. In this way the wind has been known to raise waves in the platform of a light suspension bridge.

In the case of a bridge truss it is important that the time of oscillation produced by a load acting impulsively should not exceed a certain amount, in order that the oscillations may not be cumulative.

In highway bridges it is especially the measured step of pedestrians which gradually augments the amplitude of the oscillation when the time of oscillation is equal to or greater than the time of a step, which may be assumed as from 0.6 to 0.7 second. If the time of oscillation of the structure is greater, it can adjust itself to the time of a step by the formation of centers of oscillation. The result is that by the gradual accumulation of energy changes of form are produced which are considerably greater than those produced by an equivalent static load.

These oscillations occur not only in elastic or stiffened systems, but also in slack systems. A freely suspended heavy chain moved from the position of equilibrium in its vertical plane will assume oscillating motions which will gradually increase if new impulses occur in the time intervals corresponding to the time of oscillation or a fraction thereof. It is shown by Melan that the time of oscillation of a slack chain is materially greater than that of even a very elastic construction, which explains the well-known fact that unstiffened suspension bridges can very easily be brought into great oscillations by a few pedestrians.

Prof. Melan in his treatise on Bridge Construction deduces the equations $t = 1.806 \sqrt{f}$ and $t = 2.0063 \sqrt{u + 0.8\, u_0}$ for the times of oscillation in a slack and a stiffened system, respectively; in which t is the time of oscillation in seconds, f is the versine of the cable in meters, u_0 is the deflection in the middle due to the uniformly distributed dead load, and u is the increase of deflection due to a concentrated load in the position of rest in meters.

From these formulas it follows that the time of oscillation of a bridge structure depends only upon the magnitude of its deflection in the position of rest, no matter what may be the character or size of the structure. Hence the deflection must be kept within certain limits, in

order that the bridge may not be set in vibration by the steps of pedestrians or other regularly repeated impulses.

The time of oscillation and consequently the deflection must be the smaller the more rapidly the repetition of impulses occurs. Hence greater stiffness is required in a railroad suspension bridge than in a similar highway bridge.

As the deflection u_0 due to the uniformly distributed load is in general proportional to the fourth power of the span, it follows that the time of oscillation varies approximately as the square of the span. Therefore, this time may be diminished by fastening the structure at intermediate points to the shore or ground so as to form a number of centers of oscillation.

The occurrence of cumulative oscillations may also be prevented by employing together several systems for stiffening the cables, these systems having different periods of oscillation; and where a number of cables are assembled on each side of the bridge, the same result may be accomplished by employing different versines for the cable curves. Thus, in the bridge designed by Mr. Gustav Lindenthal for the North River Bridge Company, the stiffening is obtained by trussing between the cables and by continuous longitudinal platform girders. As these two systems have different periods the chance of cumulative oscillations is greatly reduced. In the Niagara Suspension Bridge the two systems of cables have versines of 54 and 64 feet, respectively, and consequently different periods of oscillation.

The arrangements to prevent deflections due to the moving load and the wind will receive consideration in other parts of this report.

BRACING.

The bracing required to stiffen the bridge may be conveniently considered under two heads, vertical bracing and lateral bracing.

1. *Vertical bracing.*—It is the object of this bracing to confine within definite and small limits the oscillations and deflections caused mainly by the rolling load. Various methods have been employed for this purpose, the principal of which are as follows:

(1) Stays extending from the tops of the towers to the platform.

(2) Stays extending from the bottoms of the towers to the cables.

(3) Longitudinal stiffening girders connected with the platform and extending over the whole length of the bridge.

(4) Bracing between two cables hanging in the same vertical plane.

(5) Trussing the cable on its concave side.

(6) Trussing between the cable and the platform.

Two or more of these methods of stiffening are frequently employed together in the same bridge. For example: Methods (1) and (3) are employed at the East River Bridge; method (3) at the Niagara Bridge; method (4) at the Allegheny River Bridge at Seventh street, Pittsburg; method (5) at the Point Bridge over the Monongahela at Pittsburg; method (6) at the Lambeth Bridge, England. Experience has proved all these methods to be effective, and for some of them special advantages of economy are claimed. Thus the over-floor stays of method (1) not only prevent the development of large vertical oscillations in the platform, but also relieve the suspension cables of a considerable part of the load. In method (2) the stays add to the weight on the cables instead of relieving them, and in this respect it is not as good as method (1). It has been objected, however, to the use

of stays in bridges of large span that they complicate the conditions of equilibrium, as it is difficult to adjust them so as to bear a definite portion of the stresses under the varying conditions of load and temperature. By bracing two cables—method (4)—we utilize the cables as the chords of the stiffening girder and save the material otherwise required for the chords. Still greater economy is claimed for combinations of the stiffening systems.

The simplest and most employed stiffening system is the longitudinal stiffening girder. Such girders are convenient for supports to the lateral bracing and for side guards, and at none of the existing bridges where other methods were employed was the girder entirely dispensed with as a part of the stiffening system. The girder rests upon the floor beams and is thus suspended from the cable. It does not support any load, but merely distributes it, hence it is absolutely a dead weight, adding nothing to the strength of the bridge.

The Board consider it very probable that for a given special case a lighter and better stiffening system than that supplied by the simple longitudinal platform girder could be worked out by combining the trussed cables and longitudinal girder systems, as has been done by Mr. Gustav Lindenthal in his elegant design for the North River Bridge. In applying this method, however, to wire cables carrying very heavy weights over a very long span some new questions of constructive detail will require solution, and for the purposes of a general investigation it seems best to follow the usual method of stiffening. It will, therefore, be assumed that vertical stiffness is obtained entirely by longitudinal platform girders with parallel chords.

These girders are usually of the open frame or lattice type. While their rigidity does not affect the actual safety of the cables which carry the entire dead and live load, it does determine the suitability of the bridge for railroad purposes. There is no doubt that a suspension bridge can be made as rigid for railroad trains as a bridge of any other system.

If a flexible cable be loaded ununiformly it will be depressed on the side of the heaviest load and will rise on the opposite side. It is the object of the stiffening girder to reduce the distortion of the cable to a practical minimum. There are two practical ways in which the girder may be constructed:

(1) As a continuous girder, loosely supported at the ends with reactions in both vertical directions, but permitting horizontal motion.

(2) As a girder loosely supported at the ends, as in the first case, and hinged in the middle.

For the first case the problem of equilibrium is statically indeterminate; that is, the conditions of equilibrium can not be formulated without including the elastic forces developed in the girder. In the second case we have only to deal with static forces, and the stresses in the girder can be calculated with a close degree of approximation by the simple law of the lever.

In the designing of stiffening girders the formulas given by Prof. Rankine have generally been employed. The formulas for the continuous girder are deduced in his Applied Mechanics (p. 370). The formulas for the hinged girder are given, but not deduced, in his Civil Engineering (p. 579). Rankine's methods, which are approximate in character, have been extended to a high degree of accuracy by subsequent investigators. Probably the most complete investigation of the straight stiffening girder is that of Prof. J. Melan, of the Technical High School at Brünn.

Rankine's values of the maximum bending moments for the hinged and continuous girders are as follows: Hinged girder, $M=0.0156qL^2$; continuous girder, $M=0.01786qL^2$; while Melan, by more accurate methods, obtains for the hinged girder $M=0.01883qL^2$, and for the continuous girder $M=0.01652qL^2$. It will be noticed that with Rankine's values the maximum bending moment is smaller for the hinged girder than for the continuous girder, while with Melan's values the reverse is the case.

Although the hinged girder presents very decided theoretical advantages, especially in the determination of the stresses, it has some disadvantages which have prevented its employment in any important practical case. The introduction of the middle joint has for its principal object the attainment of a static determination, but, as Melan has pointed out, the theoretical conditions can not be fully satisfied unless the girder and cable have a common joint at the middle. If the girder alone is jointed, increased bending strains must be produced in the cable directly over the joint. The arrangement of the wind-bracing becomes more troublesome, for, since the upper chords of the girders are cut, all the wind stresses must be transferred to the lower chords. The wind-bracing would doubtless be heavier than for a continuous girder.

Mr. Lindenthal has shown that while there are no temperature stresses in a three-hinged arch at the middle hinge they do exist for any change from the normal temperature in the connected half-arches. His investigation of this important question (which originally appeared in the Engineering News of March 10, 1888, and which has been revised by him for the Board) will be found in Appendix D. For the purposes of this investigation, however, there is no question that the hinged girder ought to be adopted, in order to avoid the complicated formulas which would be required in the other case. It will give results as accurate as the nature of the inquiry permits and on the safe side as regards weight of metal, which could be considerably reduced for any given case in practice by the use of continuous girders, or by other methods requiring more extended computations. The New York Board employed the same method.

It will appear hereafter that when considerable rigidity is required, as in railroad bridges, the stiffening truss becomes the greatest single element of weight and therefore its economical designing is a matter of the highest importance. The Board therefore appends to this report a translation of Prof. Melan's complete investigation of the straight stiffening girder, which it is believed is unknown to most American bridge engineers. A simpler investigation, but sufficiently rigorous for all practical purposes, covering both the hinged and continuous girders, has been attached by the New York Board as an appendix to its report.

In the case of the hinged girder the greatest distortion of the cable occurs when the moving load covers the platform from one tower to a point at a distance 0.105 L from the middle of the bridge. It is here assumed that there will be practically no temperature strains and the simple statical conditions will enable us to express with sufficient accuracy the weight of the girder in terms of the span.

The values of the bending moments and shears which will be used in determining the weight of the girder are based upon the following assumptions:

1. The stiffening girders are supposed to have a height sufficient to prevent great vertical flexure. So far as the vertical strains due to loading are concerned it is most economical to make the height as great as is practically possible, and with the hinged girder this may be done,

since changes of temperature are without material influence. The height of the girder will therefore be assumed at 120 feet, as adopted by the New York Board.

2. The tensions on the suspenders are supposed to be always equal; that is, the vertical reaction between the cable and the girder is uniformly distributed over the whole length of the span; and the tensions on the cable are assumed to be invariable. It has been shown by M. Boulongne* that in the case of the suspenders the values obtained on this hypothesis can not be in error more than 5 per cent, and for the cables the error is on the safe side, since it increases the amount of work required of the girder.

3. The effects of the elastic elongation of the suspenders, due to live load and temperature changes, are neglected. As remarked by the New York Board, these disturbances can be avoided by omitting the suspenders for a short distance next the towers.

The discussion of the values of the maximum bending moments and shears is omitted from this report, as it is given fully in Melan's investigation (Appendix E), and also in Appendix E to the report of the New York Board.

The curve of maximum bending moments covering the half girder, whose length is $\frac{1}{2}$ L, is very nearly a parabola, and its area is very nearly $\frac{2}{3} \times \frac{L}{2} \times 0.01883\, q\, L^2$. The average maximum bending moment will therefore be

$$M_m = 0.01255\, q_1\, L^2.$$

The average maximum chord-stress is found by dividing this moment by the height of the girder; the area of the cross-section of the chord in square inches is obtained by dividing this chord stress by the assumed working unit stress; and the theoretical weight of the chord per linear foot by multiplying this area by 3.4 pounds. To obtain the practical weight the theoretical weight must be increased by about 25 per cent for constructive details. Although the chords are subject to reversal of strains, the Board have assumed a unit working stress of 15,000 pounds, for reasons which will be fully explained elsewhere.

The weight of the upper chord in pounds per linear foot will therefore be—

$$\frac{0.01255 q_1\, L^2 \times 3.4 \times 1.25}{15{,}000 \times 120} = 0.00000002963\, q_1\, L^2.$$

The dimensions of the lower chord will have to be considerably increased, as it serves also as a chord in the wind girder. Assuming for computation a wind pressure of 2,250 pounds per linear foot, for reasons before given, the average maximum bending moment due to the wind will be—

$$M_{mw} = 187.5\, L^2$$

The unic working stress will be assumed at 30,000 pounds per square inch for reasons which will be given elsewhere. Remembering that the width of the track-platform between the axes of the girders is 100 feet, we obtain for the weight per linear foot of material to be added to the lower chord to take care of the wind-stresses

$$\frac{187.5\, L \times 3.4 \times 1.25}{30{,}000 \times 100} = 0.0002656\ L^2.$$

* Note sur les Ponts Suspendus, Annales des Ponts et Chaussées, 7 Série. Tome 1, 1892, p. 742.

The average total weight per linear foot of the lower chord is therefore

$$(0.0002656+0.00000002963\,q_1)\,L^2$$

The shearing stresses due to a continuous moving load are greatest at the origin of the span and diminish to 0, changing from upward to downward stresses at a distance $\frac{L}{4}$ from the origin. The average maximum shear without regard to sign for all positions from the origin to the middle pin is $S=0.1038\,q_1L$.

The theoretical weight of the web must be increased by about 50 per cent for constructive details and to provide for the transfer of weight from the upper to the lower chord. The lattice bars being placed at an angle of 45°, the weight of the web per linear foot of span will be

$$\frac{0.10381\,q_1L\times3.4\times1.5\times2}{15{,}000}=0.00007058\,q_1L.$$

For the total weight per linear foot of each stiffening girder we finally obtain by addition

$$\frac{p_2}{2}=q_1\,(0.00007058\,L+0.00000005926\,L^2)+0.0002656\,L^2.$$

The value of q_1, which is to be employed in this formula, must now be determined. It will be remembered that the stiffening girders carry no weight, not even their own. They simply distribute the inequalities of the live load and limit the deflections in the cables and floor system. The girders must be dimensioned with special reference to those positions of the live load which correspond to the greatest deflection. The maximum bending moment in either half-girder corresponds to a continuous live load extending from the origin towards the middle of the span and covering a distance 0.395 L. For all spans up to 2,957 feet (equal to the maximum train length divided by 0.395), the single freight train on each track (all six trains being supposed to advance together with their engines abreast) will produce the maximum bending effects, the length of such a train being 1,168 feet. For greater spans it is evident that the maximum effect will not be thus produced. Accordingly, for spans greater than 2,957 feet we should divide the maximum train load by 0.395 L to obtain the live load per linear foot of span to be used in determining the weight of the stiffening girders.

The weight of the train is 3,060,000 pounds; hence for each girder we obtain

$$q_1=\frac{3060000\times3}{0.395\,L}=\frac{23240506}{L}$$

and by substitution in the preceding equation

$$\frac{p_2}{2}=1640.3+1.377\,L+0.0002656\,L^2$$

The total girder-load per linear foot for the whole bridge will therefore be $p_2=3281+2.754\,L+0.0005312\,L^2$.

This formula leaves entirely out of consideration the fact that part of the live load is taken up directly by the cables, yet it is certain that a considerable part of the action of the live load may be thus absorbed. In his reconstruction of the Niagara Suspension Bridge, Mr. L. L. Buck, the engineer in charge, provided for a maximum deflection of 15 inches in 500 feet, and thus reduced the value of $2q_1$ in his formulas from 0.8 ton to 0.6 ton. Mr. W. Hildenbrand, in his reports relative to a pro-

posed suspension bridge across the Hudson River, which are appended to the report of the New York Board, provides for a maximum deflection giving a grade not exceeding 1 per cent. He thereby reduces $2q_1$ from 9,000 pounds to 7,800 pounds. The Board do not doubt that within narrow limits a certain degree of flexibility is an advantage to the bridge. Deflections in a system of stable equilibrium do not impair the safety of the structure as they do in an unstable system like the upright arch, and they may exert a very beneficial influence in modifying the dynamic effects of a rapidly varying live load. On the other hand, it is to be remarked that any increase in the grade of the track-platform is accompanied for fast trains by a certain increase in the dynamic action of the live load.

The proportion of live load absorbed by the cables increases as the catenary becomes flatter, but the cables must be made heavier. It is not easy to determine satisfactorily the resultant effect of these deflections, and in order to be on the safe side the Board make no allowance for them in this investigation.

Lateral bracing.—The principal duty of the lateral or sway bracing is to resist the action of the wind. The top lateral system is a light riveted lattice connecting the top chords of the stiffening girders. Since these chords are cut at the middle, the entire work of resisting wind pressure is done by the bottom lateral system. The top system, in conjunction with the cross-frames and hangers in a vertical plane above each cross-girder, serve simply to transfer the wind stresses and a portion of the load to the bottom system. We may assume for the weight of the top system 500 pounds per linear foot, and for the cross-frames and hangers, 1,920 pounds per linear foot, as computed by the New York Board. These weights may be considered constant for all values of the span within the limits of this investigation.

In the bottom system the chords are the bottom chords of the stiffening girders, in the dimensioning of which the wind stresses have already been provided for. The cross girders of the floor system form the lateral struts, and the diagonals are strained in tension. It only remains to determine the weight per linear foot of the diagonals. The depth of the wind-girders (d) is 100 feet, and the theoretical panel length (b) is 120 feet. The number of panels (n) on each side of the middle point is therefore $\frac{L}{2b}$. The theoretical panel load is 2,250 b; the assumed unit stress is 25,000 pounds; and 25 per cent is added for constructive details. The weight of the diagonals per linear foot of span will then be

$$\frac{2 \times 1.25 \times 3.4 \times 2250\, b\, L^2\, (b^2 + d^2)}{25000\, d\, L \times 4\, b^2} = 0.3889\ L.$$

For the weights in pounds per linear foot of the lateral or sway bracing, including the cross-frames and hangers, we therefore have $p_3 = 2420 + 0.3889\ L$.

WORKING STRESSES.

In determining the weights of the two most important members of the bridge—the cables and the stiffening girders—the Board have assumed working stresses which are greater than those generally adopted in truss or arch bridges of moderate span, and which, therefore, require explanation.

The most approved formulas for the determination of working stresses are based upon the experiments of Herr Wöhler, made for

the Prussian Ministry of Commerce, and published at Berlin in 1870.* These experiments not only confirmed the earlier result obtained by Sir W. Fairbairn and others, that with repeated applications of a load a bar breaks under less than its static breaking load, but they also showed that the breaking load varies inversely with the difference between the maximum and minimum stresses. Furthermore, it was found that a bar may be broken by a still smaller fraction of the static breaking load if it is alternately strained in opposite directions, the stress alternating between a positive and a negative quantity.

The principal formulas representing these results are based upon two radically different interpretations of the observed facts. In one case it is assumed that the repeated alternations of stress produce an actual weakening of the material which has been called "fatigue." This view is represented by the Launhardt-Weyrauch formulas, which are as follows:

For stress in one direction—

$$a = u\left(1 + \frac{t-u}{u}\phi\right)$$

For alternating positive and negative stresses—

$$a = u\left(1 - \frac{u-s}{u}\phi\right)$$

in which $\phi = \dfrac{\text{min. s}}{\text{max. s}}$ = the ratio of the least to the greatest stress; a = breaking strength under the assumed conditions, which is to be divided by the factor of safety (generally 3); t = breaking strength under a static load; u = the limiting strength (Ursprungsfestigkeit), measured by the greatest load the bar will bear with an indefinite number of alternations between 0 and u without reversal; and s = vibration-resistance, which is the limiting strength for alternations of equal magnitude with reversal.

In the other case it is assumed that the alternations produce no change whatever in the molecular condition of the material, but that the increased effects are produced entirely by an increase in the stresses due to dynamic action, the stress being equal to the load only when all the forces acting are in static equilibrium. This view is represented by the so-called dynamic formula, which is

$$a = \text{max. S} + \eta\,(\text{max. S} - \text{min. S})$$

In this formula η is a coefficient depending upon the violence and time-rate of the load-changes.

The Launhardt-Weyrauch formulas are based entirely upon Wöhler's experiments, and do not take into consideration variations in the rate or violence of the dynamic action. Prof. Fidler says that in these experiments the load was applied about four times per minute.

Prof. Fidler has shown that when the alternations are rapid (for which case $\eta = 1$) the dynamic formula represents Wöhler's experiments as accurately as the Launhardt-Weyrauch formulas. No satisfactory determination, however, has been made of the value of η for the various cases occurring in practice. In the case of cross-girders and vertical suspenders, which receive the full action of the elastic vibrations due to a sudden imposition of the load, Prof. Fidler assumes $\eta = 1$, and he adopts the same value for the diagonals of the web bracing. For the flanges of a girder or in the principal members of an arch or suspension bridge, in which the stress-changes take place more gradually, he recommends a reduced value bearing some unknown

* Uber die Festigkeitsversuche mit Eisen und Stahl.

relation to the length of span. He considers the value $\eta = \frac{1}{2}$ to be large enough for all spans down to about 100 feet.

It is not possible in this report to make an extended comparison of the merits of these formulas. It may be remarked, however, that the effects of variations in dynamic action certainly play an important part in the determination of ultimate strength, although there are as yet no experiments showing how far this strength is affected by the frequency as distinguished from the mere number of the stress-changes, nor whether a period of rest after "fatigue" restores strength. These effects of dynamic variations, which are entirely unrepresented in the Launhardt-Weyrauch formulas, really exist, and are of special importance in the theory of suspension bridges. A clear and able discussion of the whole subject will be found in Chapter XIII, of Prof. Fidler's Practical Treatise on Bridge Construction.

The Board have adopted for the cables a working stress of 60,000 pounds, which is one-third of the static breaking load. Prof. Melan says that, owing to the lack of experiments with steel wires, we can consider the laws of Wöhler only so far as to allow for large spans a somewhat greater value of the working stress. For ordinary spans he adopts a working strength of about one-fourth the ultimate strength of the wire. The Board believe that a safety factor of 3 is amply sufficient to cover both the effects of variations in stress and the imperfections of manufacture and adjustment in the cables. As regards variations in stress, it is to be remarked that there are no reversals, the wire being always in tension; that considerable deflections correspond to relatively slight changes in stress; and that the stresses are slowly and gradually applied, and well within the high elastic limit.

This latter point is of special importance, for it is probable that Wöhler's law of reversals does not hold good for stresses well within the elastic limit. For example, in the balance spring of a watch, tension and compression succeed each other some 150,000,000 times in a year, and the spring works for years without apparent injury.* In this connection it may be remarked that, although cables which have been long in use have been frequently examined, no deterioration of strength which could be attributed to variations of stress has ever been discovered. If we use the Launhardt formula, we are justified in making u very nearly equal to t.

If we employ the dynamic formula, the factor max. S—Min. S will be very small, for the reasons just given. As for the coefficient η, we only know that it diminishes as the span increases, and, according to Prof. Fidler, it need not be greater than $\frac{1}{2}$ for a span as small as 100 feet; it must therefore be a very small fraction for spans as large as those now under consideration. The variation-term of this formula will probably be so small that it may be safely neglected.

As regards imperfections of manufacture and adjustment, which are covered in general practice by the safety factor 3, the following points are to be noted. The uniformity of strength is greater for wire than for any other form of steel. The process of manufacturing a wire cable is in itself a test of the material and insures a more nearly uniform distribution of stress over the cross-section than can be obtained in any other structure formed of a very great number of parts. If a factor of 3 is sufficient to cover the defects of material and construction of a riveted bridge-member, a somewhat less factor ought to be sufficient for a wire cable.

* Prof. Ewing, Strength of Materials, Enc. Brit.

For the reasons above given, the Board are of the opinion that a safety factor of 3 is sufficient, and have therefore adopted 60,000 pounds per square inch as the working stress for the cables.

The New York Board adopted the same working stress, giving as their reason that this is "the same proportion of the ultimate strength that the 20,000 pounds adopted in the cantilever structure bears to the probable strength of eye-bar steel."

The Board have adopted 15,000 pounds per square inch for the working strength of the stiffening girders. The New York Board limited the stresses due to a moving load to 12,500 pounds, because there is a reversal of strains, but allowed the stresses from the combined effects of moving load and wind to run up to 22,500 pounds. The reasons of this Board for adopting a higher working stress in the stiffening girders are as follows:

Although there is a theoretical reversal of strains, it will rarely and perhaps never occur with the maximum stress, since this would require six of the heaviest freight trains, abreast of each other, to cross and recross the bridge, first in one direction and then in the other. This would probably never happen on a bridge devoted principally to passenger traffic, and it could be prevented by the simplest police regulations. Again, the lower chords of the girders have been made of sufficient strength to resist the combined maximum stresses of the live load and the wind; but the maximum chord-stresses could never occur at the same time, since with the maximum wind pressure no trains could cross the bridge. Some allowance has been made for this by the adoption of a working strength of 30,000 pounds for that part of the material added to resist wind. The only duty of these girders is to distribute the live load and thus prevent inconvenient deflections. It is not necessary to give them the margin of strength which they would require if they were essential to the stability of the bridge.

The Board are of the opinion that the great distinction between the stable equilibrium of a suspension bridge, which can not break down from the failure of any stiffening member, and the unstable equilibrium of a truss, arch, or cantilever bridge, in which the failure of a member may involve the collapse of the entire bridge, ought to receive full recognition in the adoption of unit stresses and safety factors. The weight of the stiffening girders constitutes the most important single element in the suspended weight of the bridge, being for the maximum span about one-half the entire permanent load. It should be made no greater than is absolutely necessary, for the structure ought not to be kept under a continuous stress to provide a larger margin for stresses which may never occur. The Board believes that the working stresses adopted are amply sufficient for the members of the bridge.

TOWERS.

The weight of the towers forms no part of the suspended load, and therefore is only indirectly connected with the question of the maximum span. There is, of course, a practical limit to the height to which the towers can be carried, and the relation between their cost per vertical linear foot and the cost of the suspension system per linear foot of span will be an important element in determining the most economical versine for the cables.

The towers will be supposed to be formed of steel columns braced together, and will start from the upper surface of the masonry, 165 feet below the lowest point of the cables. An empirical formula, giving

the approximate weight of metal (W_t) in the towers, has been deduced by Mr. Lindenthal from various estimates of designs for suspension bridges, as follows:

Since the section of the cable is throughout the same, the tangent to the cable at the tower in the end span should intersect a horizontal line tangent to the cable curve at a distance from the axis of the tower not greater than that of the intersection of the similar tangent in the middle span; hence the end spans should each be at least one-fourth of the main span, and the entire length of the bridge from face to face of anchorage should be at least 1.5 L. Subject to this condition, the end spans should be made as short as convenient to save cable weight. This is also important, when the backstays carry any directly suspended load, because the bending moments from the live load in their stiffening girders may otherwise become greater than in the main span. In the present investigation, however, the loads in the side spans are supposed to be supported from beneath, and the backstays have simply to transmit the suspended load of the main span to the anchorages, the pressure on the top of each tower being equal to the total dead and live load of the main span.

Let L_a = length of the bridge exposed to wind pressure reacting laterally on the towers, in this case equal to L.

h_t = height of the metallic portion of the towers from bedplate to cable bearing.

W_s = suspended dead load plus maximum live load per unit of span.

R_t = reaction at top of towers, = 2 L W_s (for both towers).

w_1 = weight of steel per linear foot of square inch cross section = 3.4 pounds.

S = factor of safety. This will be assumed as 3.

u = ultimate strength of steel per square inch, corresponding to S = 1.

a = coefficient of practice, including stairways, housings, cable bearings, etc., deduced from actual designs = 1.65 (Lindenthal).

Steel having an ultimate strength of from 90,000 to 100,000 pounds per square inch, and an elastic limit from 56,000 to 60,000 pounds, is considered by Mr. Lindenthal more suitable and economical for heavy towers than a forgeable or punchable steel, with an ultimate strength of 60,000 pounds. All rivet holes in such high steel must be drilled and not punched and reamed.

The metal in the towers is proportional to the reaction R_t and the height h_t. The weight of metal in the towers, exclusive of bracing, will therefore be $\frac{R_t\, h_t\, s\, a\, w_1}{u}$.

The towers require bracing against wind pressure and bending from temperature changes in the cables. The metal in the braces will be proportional to the square of the height of the towers and to the length L exposed to wind pressure and temperature changes; hence the weight of the bracing in tons will be L h_t^2 S b, in which b is the coefficient of proportional weight deduced from actual designs = 0.001 (Lindenthal).

The weight of the towers will therefore be

$$W_t = \frac{R_t\, h_t\, s\, a\, w_1}{u} + L\, h_t^2\, s\, b = h_t\, S\, \frac{(R_t\, a\, w_1 + L\, h_t\, b)}{u}$$

Making $s=3$, $w_1=3.4$, $a=1.65$, $b=0.001$, $R_t=2\, W_s L$, and $u=60000$, we obtain for the weight of the towers in pounds

$$W_t = \frac{L\, h_t\, (0.187\, W_s + h_t)}{333}$$

This is on the assumption that the towers are constructed on the plan followed by the New York Board, so that the cables may be arranged side by side, and that steel of an ultimate strength of 60,000 pounds per square inch is employed in their construction.

BACKSTAYS.

The length and consequently the weight of the backstays will depend entirely upon the arrangement of the end spans, and this will in every case be determined by the local conditions. If l represents the length of a single backstay, the total weight (W_b) of the backstays for the whole bridge will be $W_b=2\ l\ w'$ in which w'=the weight of all the cables per linear foot.

If the backstays intersect the horizontal plane, tangent to the cable-curve at a distance $\frac{1}{4}$ L from the axis of the towers (which is the most economical arrangement so far as the total amount of cable metal is concerned), the length of each stay from the floor level to the top of the tower (the floor being considered horizontal and 60 feet below the lowest points of the cables) will be $5\ (\frac{L}{8}+60)$, to which should be added a constant length of about 100 feet to carry the end of the stay to its point of connection with the anchor chain, which should be well below the floor level. For this case the formula becomes

$$W_b=(468.3+0.559\ L)\ w'=(449.6+0.537\ L)\ w.$$

ANCHOR CHAINS AND PLATES.

The anchor chains are formed of steel eye bars and connect with the cables outside of the masonry of the anchorages, and with bearing plates of rolled steel at their lower ends. They are proportioned for a stress of 20,000 pounds per square inch with an allowance of 40 per cent for the weight of pins and constructive details. The tension on each backstay is 18,960,000 pounds. The weight of the anchor-chains per linear foot for each backstay will therefore be

$$\frac{18.960,000\times3.4\times1.4}{20,000}=4,512 \text{ pounds.}$$

The length of each chain may be assumed as 200 feet. The weight of chains for each backstay will therefore be 902,400 pounds. The weight of steel in each anchorage plate may be assumed as 100,000 pounds, making the total weight of anchorage metal for each back-stay 1,002.400 pounds. If n represents the number of standard cables in the bridge, the total weight of the anchorage metal (W_a) will be $W_a=2,004,800\ n$.

In this formula no deduction of weight is made for the diminution of the tension due to the friction of the chains on their supports.

For the bridge of maximum span with 16 standard cables we have W_a=32,076,800 pounds=16,038.4 tons.

MASONRY AND FOUNDATIONS.

Anchorages.—As the anchorage masonry acts merely as a weight, an inexpensive class of masonry can be used everywhere except, perhaps, in the immediate vicinity of the bearings of anchorage cables and plates. The foundations need go no deeper than necessary to obtain a

soil giving sufficient resistance to horizontal sliding, and therefore will be of comparatively simple and easy construction.

Tower foundations.—The lower portion of the towers above ground and all that below ground will naturally be built of masonry, and may be all treated as constituting the tower foundations. Being proportioned directly to the weight which they have to carry, such foundations for suspension bridges differ from those of other bridges only so far as affected by the great height of the towers proper and their consequent great weight and leverage; and therefore are like other foundations except that they must be given more cross-section, more care, and a better footing upon their beds.

In proportioning such piers, the New York Board adopted the following limits of stress:

The pressure between the metallic bed-plates and the top of the masonry should not exceed 20 tons to the square foot. The pressure within the masonry and on the foundation should not exceed 10 tons to the square foot; but in determining these pressures, the weight of materials displaced by the pier is to be deducted.

The New York Board remark that "while these pressures have been exceeded in some structures, they are higher than in usual practice, and call for masonry of good quality and of more than ordinary cost."

The method of foundation construction will depend greatly upon local considerations. For a bridge of maximum span these foundations should rest upon solid rock, if possible, and at least upon hard, incompressible impermeable soil. Modern methods have already established foundations at a level of 162 feet below the water surface, and provide means for going still deeper, if necessary, and for obtaining a properly leveled surface in the rock when found; so that the question of foundations affects to-day only the economy and not the engineering practicability of bridge construction.

THE ENGINEERING PROBLEM OF MAXIMUM SPAN.

It is now proposed to investigate the maximum length of span practicable for a suspension bridge entirely from an engineering point of view, leaving the question of the relation between traffic capacity and cost of construction for subsequent consideration.

If we suppose the cable-curve to be referred to rectangular axes through the lowest point as an origin, we have from the construction of the funicular polygon—

$$\frac{dy}{dx} = \frac{1}{Q} \int P \, dx$$

in which Q represents the constant horizontal tension at any point of the polygon, and P the total suspended load per linear foot of span. The units are the pound and the foot.

The load P is composed of the weights per unit of span of the live load, track-platform, bracing, cables and suspenders, some of which vary slightly with x; but the Board has found by a careful analysis that even with the unusual weights and spans considered in this investigation, the error involved in the assumption that P is constant is too small to be of any practical consequence. It is therefore assumed that the load is uniformly distributed, in which case we obtain by integration for the equation of the cable-curve $y = \frac{p x^2}{2Q}$ which represents a par-

abola. If L represents the span and R the ratio of the span to the versine of the cable we obtain $Q=\frac{P\ L\ R}{8}$.

The greatest stress on the cables is at their highest points, and for this maximum tension the strength of the cables must be proportioned. Its horizontal component is the constant horizontal tension Q and its vertical component is the weight on the half span, $\frac{P\ L}{2}$. If T represents this maximum stress we obtain

$$T=\sqrt{\frac{P^2 L^2}{4}+Q^2}=\frac{P\ L}{8}\ \sqrt{R^2+16}=\frac{(p'+w)}{8}\ L\ \sqrt{R^2+16}.$$

If we represent by L_1 the limiting span, that is, the span at which the cable will carry its own weight with a given stress per unit of cross-section without carrying any other load whatever, we obtain from the above equation, by making $p'=O$ and $L=L_1$

$$T=\frac{w\ L_1}{8}\ \sqrt{R^2+16}$$

hence

$$\frac{p'+w}{w}=\frac{L_1}{L}$$

From the above equations we obtain

$$L_1=\frac{8\ T}{w\ \sqrt{R^2+16}}.$$

For metallic towers and large spans the value R=8 will be generally about the most economical value for the ratio of the span to the cable versine, when the cost of the foundations is taken into consideration. If we make R = 8 and substitute for T the working strength per square inch of the material of the cable, which we have assumed as 60,000 pounds, and for w the weight of a linear foot of the cable measured horizontally and having a cross-section of one square inch, which is 3.54 pounds, we obtain $L_1=15160$ feet;
and

$$\frac{p'+w}{w}=\frac{15160}{L}$$

The values of p' and w in pounds, as determined previously, are as follows:

$p'=13605+27764726\ L^{-1}+3.24906\ L+0.00055335\ L^2+0.000000003\ L^3$
$w=17917.$

Substituting these values and reducing we obtain

$31522\ L+3.24906\ L^2+0.00055335\ L^3+0.000000003\ L^4=243856994,$

the solution of which gives for the practical maximum span

$$L=4335 \text{ feet.}$$

This span is measured between the highest points of the cables at opposite ends of the bridge.

The Board consider this a conservative value of the maximum span, as it is based upon assumptions well within the limits of theory and experience.

The elements of the bridge, deduced by the preceding formulas, are as follows:

Span between tops of cables	feet..	4,335
Height of towers above masonry	do...	707
Number of cables		16
Diameter of each cable with wrapping	inches..	21½

Suspended weight per linear foot of span.

	Pounds.
Live load	6,353
Platform	7,200
Stiffening girders	25,202
Wind bracing	4,106
Cables	17,917
Cable wrapping	433
Suspenders	1,445
Total suspended weight per linear foot	62,656

= 31.328 tons.

	Tons.
Suspended weight for whole middle span =	135,807
Weight of backstays	24,882
Weight of anchor chains and plates	16,038
Weight of towers	57,172
Total weight of metal in the bridge	233,899

THE RELATIONS BETWEEN SPAN, TRAFFIC AND COST.

In the preceding pages the Board have determined to the best of their ability the maximum span practicable for a suspension bridge from a purely engineering point of view. Their instructions further require them to investigate the maximum span "consistent with an amount of traffic probably sufficient to warrant the expense of construction." This involves the consideration of two subjects; the cost of construction and the traffic capacity of the bridge.

The cost of a suspension bridge can not be determined simply as a function of the span and traffic. In the construction of every such bridge there are elements of cost which depend almost entirely upon local conditions, and can not be estimated even with the roughest approximation until these conditions are fully known. For example, the cost of the piers will depend upon the depth of the solid foundation below the bottom of the stream or the surface of the ground; the cost of the towers and anchorages will depend upon the height at which it is necessary to elevate the roadway above the water surface, which again will depend upon the character of the river navigation; the cost of spaces for anchorages, approaches and terminal facilities will depend upon the local land values.

By examining the detailed costs of several very large bridges, it is found that these indeterminate local elements constitute on the average more than 60 per cent of the cost of such bridges in cities. It has been stated that in the case of the New York and Brooklyn Bridge, the cost of the bridge structure proper was only one-third of the expenditure for the entire work. In the case of the suspension bridge to cross the Hudson at New York, estimated for by the New York Board, the local elements determine about 54 per cent of the whole estimated cost, although the cost of approaches, terminal facilities and land are not included.

The determination of any relation between traffic capacity and the cost of construction warranted thereby is equally difficult. It may, of course, be assumed that the bridge of maximum span will be constructed only in a locality where the conditions of commerce justify the belief

that the traffic capacity of the bridge will be fully utilized. But in the general case it is impossible to determine what charges the traffic will bear. Moreover, the construction of such a bridge might be desirable even if the traffic were not likely to give a sufficient return for the vast sum invested. By the combined action of railroad companies, such a bridge might be built for the general benefits resulting from increase and facility of traffic, even though it might fail to earn directly a reasonable interest on its cost; and the enterprise might be assisted by adjacent cities, as was done in the case of the New York and Brooklyn Bridge.

But while the Board have been unable to arrive at definite conclusions in the general case, they believe that much may be learned from the study of the problem as limited by the conditions of a special locality, and the material for such an inquiry is furnished by recent investigations in connection with proposed bridges across the Hudson River at New York.

It is said that the number of ferry passengers crossing from New Jersey to New York City now exceeds 85,000,000 per year, and the passenger and freight traffics are growing rapidly. It can scarcely be doubted that a bridge in this locality would be used to its full capacity. Such a bridge would, however, be employed principally for passenger traffic, the facilities for moving freight on floats at water level to any point on the water fronts being ample and convenient.

The Hudson River at New York forms the most important part of the interior harbor. Its mid-channel depth of at least 49 feet, and its clear width of at least 2,800 feet between pier-head lines, make it one of the finest roadsteads in the world. It is navigated by an enormous commerce. Strong protests against its obstruction by a pier in the channel have been made by the commercial interests of the port. The least objectionable location for such an obstruction would be not far from the middle point, between the pier-head lines, where it would divide the upstream and downstream traffic, but this location is prohibited by the great depth to a firm foundation.

The New York Board reported that it is safe and practicable to cross the river with a single span, and estimated the cost of a suspension bridge for that purpose, its New York pier being between Fifty-ninth and Sixtieth streets, at $30,743,000. This is the estimate for their *Lighter Structure*, but it provides for a bridge amply sufficient for the purposes for which it is intended. Moreover, the estimate was made for the purposes of comparison, and the report of the Board distinctly states that it is not to be taken as an absolute estimate of cost. This Board considers this estimate perfectly satisfactory for the purpose for which it was made, but they think it desirable to determine a minimum as well as a maximum estimate, to show the variations to which such estimates are liable and how much they are affected by legitimate differences in the assumptions upon which they are based. An estimate has therefore been made on the following assumptions:

The cost of structural steel is taken at 4 cents per pound, in accordance with the views of a majority of the New York Board, as indicated in their report.

The cost of wire work is taken at 7 cents per pound, which is based upon prices given by leading manufacturers and upon actual experience in the case of the New York and Brooklyn Bridge.

The weights of metal are determined by the formulas given in this report.

The bridge is supposed to be located near Sixty-ninth street, New

York, and the cost of the substructure is assumed to be the cost at the lower location (between Fifty-ninth and Sixtieth street), as estimated by the New York Board, less $2,900,000, which they state would be saved by adopting the upper location. The minimum estimate is as follows:

Structural steel:		
Suspended weights	pounds	90,870,000
Towers	do	52,313,000
Chains and anchor plates	do	18,324,000
Total	do	161,507,000
At 4 cents per pound		$6,460,280
Wire work:		
Main cables and wrapping	pounds	30,358,000
Backstays and wrapping	do	22,738,000
Suspenders	do	3,222,000
Total	do	56,318,000
At 7 cents per pound		$3,942,260
Cost of superstructure		10,402,540
Cost of substructure		11,784,000
Total cost of bridge		22,186,540

The final plans for a work of such magnitude would only be adopted after the most extended theoretical and experimental investigations, and the estimated cost would undoubtedly be much reduced by such studies. Assuming the most favorable location and the most competent engineering management, the Board believe that $23,000,000 is a reasonable estimate for a six-track railroad suspension bridge 3,200 feet long, and they consider the amount of traffic which such a bridge would accommodate sufficient to warrant the expense of construction. They believe, however, that the bridge should be so constructed that its capacity can be readily increased, and with the suspension system this can be provided for by giving suitable dimensions to the towers and anchorages.

If sufficient inducements were offered to competent engineers to prepare competitive designs and estimates for a single-span bridge at this locality, the Board do not doubt that perfectly satisfactory plans would be obtained within the limit of cost of the estimate given above.

The Board desire to express their obligations to Mr. Gustav Lindenthal, C. E., Mr. W. Hildenbrand, C. E. and Mr. L. L. Buck, C. E. for information and valuable suggestions.

The following appendices accompany this report:

APPENDIX A.—Orders and instructions.
APPENDIX B.—Correspondence with wire manufacturers.
APPENDIX C.—Wind pressure.
APPENDIX D.—Temperature Strains in Three Hinged arches, by Gustav Lindenthal, C. E.
APPENDIX E.—The Theory of the Stiffening Girder, by Prof. J. Melan.

Respectfully submitted.

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REPORT

OF

BOARD OF ENGINEERS

UPON

NEW YORK AND NEW JERSEY BRIDGE.

Extract from act of Congress entitled "An Act To authorize the New York and New Jersey Bridge Companies to construct and maintain a bridge across the Hudson River between New York City and the State of New Jersey," approved June 7, 1894.

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That * * * the President shall appoint a board, consisting of five competent, disinterested, expert bridge engineers, of whom one shall be either the Chief of Engineers or any member of the Corps of Engineers of the United States Army, and the others from civil life, who shall, within thirty days after their appointment, meet together and, after examination of the question, shall, within sixty days after their first meeting, recommend what length of span, not less than two thousand feet, would be safe and practicable for a railroad bridge to be constructed over said river, and file such recommendation with the Secretary of War, but it shall not be final or conclusive until it has received his written approval. In case any vacancy shall occur in said board, the President shall fill the same. The compensation and expenses of said board of engineers shall be fixed by the Secretary of War and paid by the said bridge companies, which said companies shall deposit with the Secretary of War such sum of money as he may designate and require for such purpose: *Provided, always,* That nothing herein contained shall be construed as preventing the said board of engineers from meeting, investigating, and filing their recommendation after the expiration of said time herein mentioned.

MEMBERS OF BOARD APPOINTED BY THE PRESIDENT.

C. W. Raymond,
Major, Corps of Engineers, U. S. Army.
Mr. G. Bouscaren.
Mr. W. H. Burr.
Mr. Theodore Cooper.
Mr. Geo. S. Morison.

REPORT

OF

BOARD OF ENGINEERS ON NEW YORK AND NEW JERSEY BRIDGE.

OFFICE OF THE CHIEF OF ENGINEERS,
UNITED STATES ARMY,
Washington, D. C., September 29, 1894.

SIR: Referring to report, dated August 23, 1894, by the Board appointed under the provisions of the act of Congress of June 7, 1894, to consider length of span of proposed bridge across Hudson River between Fifty-ninth and Sixty-ninth streets, New York City, I beg to recommend that 500 copies of the report of the Board be printed at the Government Printing Office.

Very respectfully, your obedient servant,

THOS. LINCOLN CASEY,
Brig.-Gen., Chief of Engineers.

Hon. D. S. LAMONT,
Secretary of War.

Approved:

DANIEL S. LAMONT,
Secretary of War.

REPORT OF BOARD OF ENGINEERS.

NEW YORK, *August 23, 1894.*

SIR: The Board of Bridge Engineers appointed by the President under the act of Congress, approved June 7, 1894, authorizing the New York and New Jersey Bridge companies to construct a bridge across the Hudson River, met at the Army Building, New York City, on Monday, June 25.

Your Board organized by electing Maj. C. W. Raymond, Corps of Engineers, U. S. A., chairman, and Mr. Cooper, secretary.

Your Board began the examination of the question submitted to them and gave it careful and continuous consideration up to the present time; they have held 29 regular meetings.

Your Board personally visited and examined the site of the proposed bridge as defined by the act.

They applied to the New York and New Jersey Bridge companies for their surveys, borings, plans, estimates, and such other data as referred to the proposed bridge. The surveys and borings furnished

upon this application having been made for a former location not within the limits stated in the act, other surveys and borings upon lines within these limits were made at the request of the Board.

The New York and New Jersey Bridge companies, by their engineering representative, Mr. Charles Macdonald, presented plans prepared by the Union Bridge Company for the proposed bridge, with a statement as to cost, estimated traffic, and other data, which are given in Appendix B.

During the sessions public hearings were given to a special committee of the New York Chamber of Commerce, through its chairman, Mr. Gustav H. Schwab, and its engineering counsel, Mr. W. Hildenbrand, and also to Mr. G. Lindenthal, chief engineer of the North River Bridge Company. Their statements will be found in Appendixes C and D.

The duties of your Board as prescribed by the act are to "recommend what length of span, not less than 2,000 feet, would be safe and practicable for a railroad bridge, to be constructed over said river." The act provides that this bridge "shall not be located below Fifty-ninth street, New York City, nor above Sixty-ninth street, New York City." Your Board, therefore, understand their duties to be to recommend what length of span, not less than 2,000 feet, would be safe and practicable for a railroad bridge to be constructed over the Hudson River between Fifty-ninth and Sixty-ninth streets in the city of New York.

In making comparative estimates, your Board selected a location midway between Fifty-ninth and Sixtieth streets, but the difference between this location and one further north within the limits of the act has been considered so far as it affects the general conclusions.

The minimum length of span which may be considered is 2,000 feet, which your Board have interpreted as meaning 2,000 feet in the clear. The maximum length of span would be a clear span between the pier-head lines, this distance varying from 3,130 feet at Fifty-ninth street to 3,080 feet at Sixty-ninth street.

The objections which have been raised to a pier in the river apply with equal force to any pier located between the end of a 2,000-foot span and the pier-head line, the pier being objected to as interfering with the use of the river for harbor purposes rather than for through navigation. The plans submitted to your Board have located the 2,000-foot span, in accordance with the requirements of the New York charter, next to the New York pier-head line, thus placing the west pier about 1,000 feet from the New Jersey pier-head line; if the span is increased beyond 2,000 feet any injury done to the harbor by obstructing the approach to piers on the New Jersey shore would be greater than any benefit gained by increased width of channel span. Your Board have considered that navigation would not be benefited by making a span of greater length than 2,000 feet, unless such span could reach from pier-head line to pier-head line; they have therefore confined their examination to a span of 2,000 feet in the clear, as compared with such single span. It must be noted, however, that the pier-head lines are artificial and are subject to change under existing laws. The width between pier-head lines at this location is about 400 feet greater than at a point 2 miles below. A small encroachment beyond these pier-head lines could be permitted without essential harm; it would obstruct navigation no more than a vessel lying across the head of a pier; a span of 3,100 feet in the clear would meet all the requirements of a single span.

The plans submitted by the bridge companies provide for a cantilever bridge carrying six railroad tracks. This number of tracks is the least that has been proposed by any company which has contemplated bridging the Hudson River opposite the city of New York. Your Board have therefore thought it right to make estimates for a bridge furnishing this accommodation.

The fact that the river must be kept unobstructed during erection limits the plans to cantilever and suspension bridges. The plans submitted by the bridge companies provide for a steel cantilever bridge, a description of which is given in Appendix B.

A cantilever bridge is a rigid structure, subject to those changes of shape only which are due to strains; it is well adapted to railroad uses.

In the first place, your Board are of the unanimous opinion that a cantilever span of 3,100 feet in the clear could be built and would be a safe structure.

In the second place, your Board have considered that the practicability of such a structure would depend upon its cost, and to determine this practicability, have made comparative estimates of the cost of two cantilever bridges with clear spans of 2,000 and 3,100 feet, respectively. These estimates are comparative rather than absolute; the benefit of the doubt, where any exists, has been given to the longer span. The estimates include both substructure and superstructure, but have been made in round numbers and do not include the cost of tracks and other features which would be common to both plans.

A series of borings, covering virtually the limits permitted by the act, have been made by the bridge companies under the direction of Mr. C. B. Brush, C. E., at the request of your Board, to determine the character of the bottom of the river.

These borings have found rock at varying depths, but as the borings were not extended into the rock, the absolute information before your Board is that no rock exists above the reported elevation rather than that solid rock exists below it; but your Board have considered themselves justified in assuming that it is a substantial rock, suitable for foundations. The borings outside the limits of the special line considered have confirmed the accuracy of the others.

The depth to rock is about 125 feet at each pier-head line; it is about 260 feet at the site where the pier of the 2,000-foot span bridge would come; the rock rises rapidly from each pier-head line shoreward. The depth of water at the site of the river pier is about 50 feet. Under the water is a layer of mud or silt about 100 feet deep. Below this mud is a fine sand filled with fresh water under a pressure exceeding the head due to its depth.

The mud or silt is not a suitable material for the foundation of bridge piers. For the comparatively moderate weights carried by bridges of usual dimensions, the sand would be a suitable foundation. For the extraordinary weights and dimensions of the bridge authorized by the act, your Board are not satisfied that the piers would be safe unless founded on rock, and the comparative estimates have been made for rock foundations.

The lateral provisions to resist wind and the longitudinal provisions for stability during erection require a considerable base; the plans submitted by the bridge companies propose to use a pier consisting of four cylinders placed 200 feet between centers in each direction. The total reaction, including the effects of wind pressure, carried on each cylinder is estimated at 25,000 tons.

In proportioning these piers, your Board have found it necessary to adopt limits of stress. They have based their estimates on the supposition that the pressure between the metallic bedplates and the top of the masonry should not exceed 20 tons to the square foot, and that the pressure within the masonry and on the foundation should nowhere exceed 10 tons to the square foot; they consider, however, that in determining these pressures the weight of the material displaced should be deducted. The weight of masonry per cubic foot was taken at 150 pounds in air, at 87 pounds in water, at 50 pounds in mud, and at 30 pounds in sand. While these pressures have been exceeded in some structures, they are higher than usual practice and call for masonry of good quality and more than ordinary cost.

Your Board have assumed that the masonry would finish 50 feet above water, and have estimated the cost of these piers, including excavation and sinking, at $1 per cubic foot above a plane 125 feet below water, and have added 8 mills to this price for each additional foot of depth.

2,000-FOOT CLEAR-SPAN CANTILEVER.

The east pier of the bridge, with a clear span of 2,000 feet, would be immediately back of the New York pierhead line, where the rock is 125 feet below mean high water. The west pier would come in the river, where the rock is 260 feet below mean high water. The east anchorage would be within the shore line, where the rock is not more than 20 feet below mean high water, and the west anchorage would be immediately west of the New Jersey pierhead line, where the rock is 125 feet below mean high water. The site of the west anchorage calls for an anchorage span 100 feet longer than is shown on the plans submitted by the bridge companies.

The east pier would consist of four cylinders, each containing 866,000 cubic feet, and costing on the basis given above, $866,000, making for the four cylinders, $3,464,000.

At the site of the west pier the average depth to rock is not less than 260 feet. A foundation carried to rock here would be nearly 100 feet deeper than any foundation which has ever been put in. Such a foundation involves very careful consideration, and your Board believe that the additional price allowed for so much of the work as is more than 125 feet below water is none too large. Each of the four cylinders would contain 1,880,000 cubic feet, of which 1,014,000 would be more than 125 feet below water, making the cost of each cylinder $2,427,500, and the cost of the four cylinders $9,710,000.

The east anchorage pier would be founded on rock about 20 feet below mean high water, and the west pier on rock 125 feet below water. Each of these piers has been estimated on the basis of a pier finishing 150 feet above high water, 20 feet thick, and 100 feet long on top, built with a batter of 1 in 20, and founded on a caisson 40 by 120 feet for the east pier and 45 by 125 feet for the west pier. Taking the cost of the work above water at 75 cents per cubic foot, and of the work below water at $1, the cost of the east pier becomes $431,000 and that of the west pier $1,038,000.

The cost of the substructure for the bridge, with the 2,000-foot clear span, would then be:

East anchorage	$431,000
East pier	3,464,000
West pier	9,710,000
West anchorage	1,038,000
Total	14,643,000

A careful estimate prepared by the bridge companies makes the weight of the superstructure 230,000,000 pounds, including the main span, the towers, and the two anchorage spans, covering a total length of 4,120 feet. This weight has been checked, and may be taken as approximately correct. The plan was prepared for a location at Seventy-second street, where the distance between pierhead lines is 3,070 feet. At Fifty-ninth street the west anchorage span would be lengthened 100 feet, and if the bridge is kept symmetrical the whole length will be increased to 4,320 feet, and the total weight to about 240,000,000 pounds. This estimate is based on a moving load of 3,000 pounds per foot of track and on maximum working stresses of from 20,000 to 22,500 pounds per square inch, or about one-third of the ultimate strength of the material; 240,000,000, at 4½ cents per pound, would cost $10,800,000. The cost of this bridge would then be $25,443,000.

This is the cost of a cantilever bridge of the minimum span which your Board are authorized to consider, the length of the entire structure, from anchorage to anchorage, being 4,320 feet. As this plan of bridge is the one which the New York and New Jersey Bridge companies have selected as the bridge they wish to build, its cost must be accepted for present purposes as the cost of a practicable structure.

3,100-FOOT CLEAR-SPAN CANTILEVER.

The site of the east pier for the span of 3,100 feet in the clear would be the same as that for the 2,000-foot span; the site of the west pier would be the same as that of the west anchorage for the 2,000-foot span; both piers would be founded at practically the same depth, or 125 feet below mean high water.

The weight of the trusses of the long span would be about three times the weight of those of the short span, and the weight of the floor and moving load would be about one and a half times that of the short span. The total reaction on the piers would be at least two and one-half times that of the short span. On this supposition each of the four cylinders would have to carry 62,500 tons.

The piers in both bridges are so large that their volume can be proportioned directly to the weights they have to carry. This would make the volume of each pier of the 3,100-foot span bridge two and one-half times that of the east pier of the 2,000-foot span bridge. The estimated cost of the east pier of the 2,000-foot span bridge was $3,464,000, so that we may estimate the cost of each of the two piers of the 3,100-foot span bridge at $8,660,000.

The anchorage piers required for the long-span bridge need be little larger above the water level than for the shorter span. The anchorage pier on the east side would be on rock about 20 feet below mean high water; its cost would be about the same as that for the 2,000-foot span. The anchorage pier on the west side would be on rock 40 feet below mean high water, and is estimated to cost $527,000.

The total cost of the substructure for the 3,100-foot clear span bridge would then be:

East anchorage	$431,000
East pier	8,660,000
West pier	8,660,000
West anchorage	527,000
Total	18,278,000

Estimates made by this board show that the weight of the superstructure of this bridge would be approximately 730,000,000 pounds, about three times that of the shorter span bridge; 730,000,000 pounds at 4½ cents per pound is $32,850,000.

The total cost of the 3,100-foot span bridge, covering a length of 6,100 feet from anchorage to anchorage, may, therefore, be estimated at $51,128,000, though this estimate is probably too low.

The estimated cost of the 2,000-foot span bridge was $25,443,000 for 4,320 feet; to bring it into proper comparison with the longer span bridge 1,780 feet of viaduct must be added; estimating this viaduct at $1,000 a foot, the cost becomes $27,223,000. The estimated cost of the long-span cantilever bridge is $23,905,000 more than this amount.

Your Board are of the opinion that the additional cost of the long-span cantilever bridge is so great that it must be considered impracticable.

SUSPENSION BRIDGE.

A suspension bridge is another possible form of construction at this location; like the cantilever, it can be erected without false work; unlike the cantilever, it has not generally been considered well adapted to railroad uses.

It has less rigidity than the cantilever, and deflects more from the combined effect of temperature and load; the flexibility of the cables tends to cause vertical undulations of the platform under a moving load, which are more objectionable for a railroad bridge than for a highway bridge, where the live load is less concentrated and is applied less rapidly; these objections lessen in importance as the span of the bridge and the proportion of the dead to the live load increase.

In a bridge with six independent tracks the condition of railroad service approaches that of highway service, and the position of trains which will produce a maximum disturbance would be of very rare occurrence. The inclination of the platform longitudinally and transversely, arising from the undulations of the cables under the effect of moving trains, can be reduced within unobjectionable limits by a proper system of stiffening; the effect of wind on cables and platform can be taken care of by cradling the cables and by a lateral system of bracing in the platform similar to that used in truss bridges. A single railroad track suspension bridge of 850-foot span has been in continuous use, under restrictions of load and speed, for nearly forty years at Niagara; with this example before us, a six-track railroad suspension bridge of 3,100 feet clear span can not be dismissed without careful consideration.

Your Board have therefore investigated such suspension bridge with great care, and it is their opinion that it could be built and that it would be a safe structure. As this opinion may be thought a departure from general opinion as to the adaptability of the suspension bridge to railroad service, it is proper that the Board should state their reasons therefor, and explain the features of the plan adopted by them for a comparative estimate in more detail than was done for the cantilever plans.

The essential differences between a cantilever and a suspension bridge are, (1) that in place of the compression chords of the cantilever we have land anchorages built of eyebars and masonry; (2) in place of the tension chords of the cantilever we have cables built of wire of a superior grade of metal; (3) in place of the web bracing of the cantilever we have a composite system of suspenders and stiffeners.

No question can be raised as to the safe and permanent character of the anchorages if built with a sufficient factor of resistance and proper

provisions for thorough protection of the anchor chains against rusting; they have the advantage over the compression chords of the cantilever that their weight is supported directly on the ground, instead of forming a part of the dead load to be carried.

As regards safety and efficiency, the wire cables are fully equal to eyebar chords, if built with the same margin of strength; experience shows that they can be effectively protected against rusting by wire wrapping and painting; wire at least three times as strong as eyebar steel is a merchantable article, and cables made of this wire have the advantage over eyebar chords of less weight to be carried by the superstructure.

The objections made to suspension bridges arise only from the third difference. It is often claimed that a sufficient degree of rigidity can not be secured for railroad purposes, and that the stiffening members can not be properly proportioned, owing to the uncertainty which exists in the intensity of stresses due to changes of temperature and elastic deformation in the composite system. The Board has given careful consideration to these objections and believe that for practical purposes they are met in the plan selected.

Three principal methods have been employed to secure greater rigidity in suspension bridges; (1) by inclined stays extending from the top of the towers to the platform—this system was advocated and applied extensively by the late John A. Roebling; (2) by trussing the cables either with straight chords, as in the Point bridge at Pittsburg, or by a system of braces between two cables, as proposed by Mr. G. Lindenthal for his projected North River bridge; (3) by a stiffening girder fastened to the platform and extending from one tower to the other; this system is a feature common to nearly all suspension bridges, but has seldom been applied in the most approved form to give the best results. The first method is at best incomplete, as a stiffening truss must be used for the middle half of the span. The second method might prove the most economical, but its application to wire cables is still untried. Your Board have, therefore, selected the third method, that of stiffening the truss.

The suspension bridge which your Board have selected for this location would consist of a single span of 3,200 feet between saddles, thus giving about 3,100 feet in the clear, the two towers being located at the pierhead lines, and the cables being carried in straight lines from the top of the towers to the anchorages, making equal angles on each side of the towers. This form of bridge has no side spans, but the tracks would be carried on viaducts between the towers and the anchorages. While the use of cables outside the towers to sustain side spans is generally considered economical, the arrangement selected gives the least length of cable and reduces deflection from strains and temperature to a minimum.

The two towers would be located in practically the same position as the towers of the 3,100-foot cantilever. The substructure would be of masonry, finishing at the same height as the masonry of the cantilever bridge piers.

The towers themselves would be of steel and would be 570 feet high from top of masonry to saddles, or 620 feet from surface of water. For towers of this height there is no question of the economy and expediency of using metallic construction.

The anchorages would be of masonry, each located about 1,000 feet back of the towers. Both towers and anchorages would have to be founded on rock.

The cables would be of wire, and the plans have been based on cables containing about 6,000 No. 3 wires (0.259 inch in diameter). Wiremakers are prepared to furnish a wire of this size of a guaranteed strength of 180,000 pounds per square inch at moderate prices and a much stronger wire at a higher price. Your Board have adopted as the unit stress on cables made of straight wire of this character 60,000 pounds per square inch, or one-third of the breaking stress, this being the same proportion of the ultimate strength that the 20,000 pounds adopted in the cantilever structure bears to the probable strength of eyebar steel.

Your Board have estimated on a versed sine of 400 feet, or one-eighth of the span. In the East River bridge the versed sine is less than one-twelfth of the span and about the same as in other long-span suspension bridges. In the East River bridge the cables are of steel wire and the towers of masonry. With the introduction of steel towers, the economical proportions are changed, and it becomes practicable to adopt a greater versed sine than has hitherto been considered wise.

Stiffening truss.—There are several admissible forms of stiffening truss; to justify the particular form selected by the Board for their estimate it is proper to give a short explanation of its duties and mode of action.

A stiffening truss is a girder supported by the cables and extending from one tower to the other; it is fastened to the platform at the several points of suspension to the cables and it may be fastened to the towers in two ways; it may be held in the vertical direction only, anchored down as well as supported, and acting as a girder *resting* on two supports, or it may be fastened also in the horizontal direction, acting as a girder *fixed* at the ends. The Board have confined themselves to the first case, which has the advantage of greater simplicity in computation of stresses, without material sacrifice of economy.

The function of a stiffening girder is to distribute a load covering only a part of the span, over the entire span. If this function could be performed without any deformation of the girder, the distribution would be perfect and the symmetrical shape of the cables would be preserved, but as the girder deflects under the load that it carries, it exerts through the suspenders a downward pull on the cables as far as the load extends, and beyond that point the cable exerts a pull upward on the girder. If it is continuous it will take the shape of a reverse curve with its point of contraflexure at the end of the load. The strain in all the suspenders will be uniform for the whole length of span. The weight to be carried by the loaded portion of the stiffening truss will be the moving load upon it, less that carried by the suspenders.

The suspenders over the unloaded portion, where the cable tends to rise, are strained by the resistance of the stiffening truss against flexure upward. The weight per unit of length carried by the suspenders will always be equal to the live weight per unit of length multiplied by the length of load and divided by the length of span. Over the loaded portion this is the actual weight per unit of length, less the portion carried by the stiffening truss. Over the unloaded portion this represents the upward pull resisted by the stiffening truss. The upward force per unit of length, which tends to lift the unloaded portion, is therefore the assumed weight per unit of length multiplied by the length of load and divided by the length of span. The weight per unit of length carried by the stiffening truss on the loaded portion is the total weight per unit of length less that weight multiplied by length of load and divided by length of span.

When one-half of the span is loaded, the weight will be equally divided between suspenders and stiffening truss; the stresses in the chords of the stiffening truss will be one-eighth those caused by the same load extending over the whole length of span if the truss were not supported by suspenders.

The greatest stresses occur in the continuous stiffening truss when either two-thirds or one-third of the span is loaded. In the former case the loaded portion must carry one-third of the load, and the chord stresses at the middle of that two-thirds will be four twenty-sevenths of the maximum stresses at center of span if the truss were fully loaded and not supported by suspenders; in the latter case the chord stresses at the center of the loaded portion will be only two twenty-sevenths, while the chord stresses at the center of the unloaded portion will be the four twenty-sevenths, but reversed. As the two-thirds load may be placed anywhere in the truss, it follows that the chord stresses over the whole central third may be four twenty-sevenths of the maximum stress at center of span if the truss were fully loaded and not supported by suspenders.

The shearing stresses in the webs of the stiffening truss are determined by the same distribution of loads.

It must be remembered that the only stresses in the stiffening truss are those due to moving load, all dead weight being carried directly by the suspenders to the cables.

While this is the simplest explanation of the duties of the stiffening truss, it does not take into consideration all elements. The downward and upward deflection of the stiffening truss must be accompanied by corresponding changes in the shapes of the cables, but as these changes are in the direction in which the cables would move if no stiffening truss existed, it follows that the weight is not distributed equally among all the suspenders and the stiffening truss is relieved of resisting so much inequality as is taken by the cables. The elongation of the suspenders is also a slight element of disturbance, but not sufficient to be described here; an analysis of it will be found in Appendix E.

There are two other strains which the chords of the stiffening truss may be called on to resist. The first of these is due to the deflection of the cables under temperature and under load. As the stiffening truss is not supposed to carry any of its own weight, it must deflect with the deflection of the cables, and this deflection must be accompanied by the transfer of a portion of its weight to itself with corresponding stresses in its chords, these chord strains being determined entirely by the deflection. The other additional stress is due to wind pressure if the chords of the stiffening truss are made the chords of the lateral system, as in ordinary truss bridges.

The greater the depth of truss the less the chord stresses due to its stiffening duty and the greater the stresses due to deflection of cables. The wind stresses depend on the horizontal distance between the two trusses.

To avoid the strains due to deflection of cables the stiffening truss may be hinged at the center, which can be done by cutting one chord and putting a pin joint in the other. This arrangement fixes the point of contrary flexure at the center of the span under all conditions of loading, and leaves the stiffening truss free to rise and fall with changes of deflection in the cables without additional strain. As the bending stresses at the center are now eliminated, the only function of the pin joint will be to transfer the shearing stresses. With the introduction of the hinge and the fixing of the point of contrary flexure, the work

of the stiffening truss is modified, and the investigation becomes more complicated; it is given in Appendix E. The truss still equalizes the weight on all the suspenders, but the total weight carried by the suspenders is equal to the whole moving load only when that load covers one-half the span. The greatest chord stresses in either direction occur at a distance equal to 0.234 of the span from each end, and will be 0.1506 of the maximum stresses at center of a continuous span if the truss were fully loaded and not supported by suspenders. The maximum chord stress in the hinged truss is, therefore, 1.017 times that in the continuous truss, but it is a maximum only at two points instead of over one-third the span.

The continuous truss is better adapted to resist wind than the hinged truss, but its chords have to bear the additional stress due to deflection. As the hinged truss is practicable and more economical than the other, it has been used in the estimates made by your Board.

The form which your Board have selected for a stiffening truss is a riveted lattice girder 120 feet deep, the two trusses being placed 100 feet between centers. The web members are all inclined at an angle of 45 degrees, and are in eight systems, so that the truss is divided into 30-foot panels and the unsupported length of each web member is about 21 feet. The floor beams are hung from the suspenders and carry the stiffening truss, the weight of which is never entirely overcome by the action of the moving load. The top lateral system is a comparatively light riveted lattice. The whole lateral work to resist wind pressure is done by the bottom lateral system, in which the floor beams form lateral struts and the diagonals are strained in tension. Cross bracing is provided at every panel point, to sustain the floor beams at their centers and to transfer wind pressure to the bottom chord, the pull of this cross bracing being resisted by the top lateral system.

Proportioning the trusses for a moving load of 3,000 pounds per foot on each of the six tracks, the maximum chord stress at the center of the half span would be 14,461,400 pounds, and the maximum chord stress in the bottom chord at the center, taken on the basis of a wind pressure of 2,000 pounds per linear foot, would be 25,600,000 pounds. As the chords are subject to reversal of strains, your Board have limited the stresses in the chords due to moving load to 12,500 pounds per square inch in each direction, making an extreme variation of 25,000 pounds, but have allowed the stresses from the combined effects of moving load and wind to run up to 22,500 pounds, believing that, with the arrangement of cradled cables hereinafter described, the wind strains will never be anything like what has been estimated on. They have also estimated on the chord sections never being less than 400 square inches. With these conditions, the average section of the bottom chord becomes 996 square inches, and that of the top chord 905 square inches, the two averaging 950 square inches. Allowing 25 per cent excess for details, the average weight of each chord will be 4,037.5 pounds per linear foot.

The average shear in the web system will be 3,000,000 pounds, in addition to which the web system has to do a duty in transferring weight from the upper to the lower chord equivalent to a shear of 1,300,000 pounds per linear foot, making the total duty of each web equivalent to an average shear of 4,300,000 pounds. If the web is proportioned on the basis of 12,500 pounds per square inch, with an allowance of 50 per cent for details and connections, the weight of each web becomes 3,509 pounds per linear foot.

The calculated weight of the top laterals is 500 pounds per linear foot. The calculated weight of the bottom laterals, on the basis of 25,000 pounds stress per square inch, with an allowance of 25 per cent for details, is 1,150 pounds per linear foot, making a total weight of laterals 1,650 pounds per linear foot.

The floor beams weigh 90,000 pounds each, or 3,000 pounds per linear foot of bridge. The stringers weigh 1,800 pounds per linear foot of bridge. Floor beams and stringers are proportioned for a consolidated locomotive weighing, with tender, 104 tons. The total weight of the suspended superstructure per linear foot may then be taken as follows:

	Pounds.
4 chords, at 4,037.5 pounds	16,150
2 webs, at 3,509 pounds	7,018
Laterals	1,650
Cross frames and hangers	1,920
Floor beams	3,000
Stringers	1,800
Total steel per linear foot	31,538

This amounts to 100,921,600 pounds for the 3,200 feet of span. If to this we add 2,400 pounds for the weight of the ties and rails and 18,000 pounds for moving load, we have as the total weight carried by the suspenders 51,938 pounds or 26 tons per linear foot.

This stiffening truss is a very different structure from the stiffening truss of any existing bridge. It is what it purports to be, a stiffening truss, with a heavy floor system like that used in the cantilever design, and with stiff connections throughout. This stiffening truss, 3,200 feet long with its floor system, weighs two-fifths as much as the entire 4,320 feet of steelwork of the 2,000-foot cantilever bridge.

Suspenders.—The suspenders would be either wire ropes or cables of straight wires, like the main cables. They have been proportioned on the basis of a stress of 30,000 pounds per square inch of section, and on this basis, with an allowance of 20 per cent for connections, will weigh 1,425 pounds per linear foot, making the whole weight transferred to the cables 53,363 pounds. The suspenders weigh 4,560,000 pounds for the 3,200 feet.

Cables.—The average weight of the cables will be 14,792 pounds per linear foot of bridge. The total weight to be carried by the cables may therefore be taken at 68,100 pounds per linear foot, amounting to 217,920,000 pounds or 109,000 tons for the span of 3,200 feet. The versed sine assumed is 400 feet, or one-eighth of the span. The greatest strain in the cables will be next to the saddles, and will be equal to the weight carried multiplied by 1.118, amounting to 243,724,000 pounds, which, at 60,000 pounds per square inch, will require 4,062 square inches. Six thousand No. 3 wires have a total area of 316 square inches. The 4,062 square inches may be divided into 12 cables of 338.5 inches each. Your Board believe that these cables can be constructed now as easily as those of the East River bridge were at the time it was built.

The arrangement of cables which has seemed most feasible to your Board, and which has been used for the basis of these estimates, places six cables on each side, the cables being 20 feet apart on top of towers, the two cables next to the center on each side being in vertical planes, and the other cables cradled into planes which intersect in the lines of the pins which sustain the floor beams. A separate suspender reaches from each pin to every cable, the suspenders being in the same planes as the cables. Vertically the cradling of the outside cables is 100 feet

in a height of 460 feet, or 1 in 4.6. Horizontally it is 100 feet in a total length of 3,200 feet, so that the horizontal cradling of the two outside cables is 200 feet in 3,200 feet, or 1 in 16. A sufficient cradling is obtained not only to resist the entire wind pressure on the cables, but to relieve the lateral system very materially. The distance between the cables will favor simultaneous construction. The suspenders at each point will be of uniform length and will pull together. The length of the suspenders at the center of the span must be enough to allow the cables to clear each other where the attachment is made, and this places the lowest parts of the cables 60 feet above the pins. The total height of the towers above high water is made up as follows: Clearance required by law, 150 feet; camber, 10 feet; shortest suspender, 60 feet; versed sine, 400 feet; total, 620 feet.

The total length of each cable, from anchorage to anchorage, is 5,609 feet. The weight of each of the 12 cables, per linear foot of cable, including wrapping, is 1,183 pounds. The weight of the 12 cables is 14,200 pounds per linear foot, and the total weight of the cables, 79,647,800 pounds.

Towers.—The weight transferred by the cables to each tower is 218,000 pounds. The towers are 570 feet high from top of masonry to saddles. As these towers are only in compression and the members so large that they may be treated as short compression members, a stress of 20,000 pounds per square inch at the top is permissible. This requires 10,900 square inches of section. The weight of each tower with an allowance of 80 per cent for details and connections would be 38,023,560 pounds or 76,047,000 pounds for both towers. The total weight to be carried on the lower part of the tower would be 128,000 tons, making a pressure of less than 24,000 pounds per square inch at the base of the steel columns, which will be very slightly increased by the wind pressure and by the horizontal deflections at the top of the towers if the saddles do not move freely.

Anchor chains.—The cables are carried in straight lines from the saddles to the anchorages, each anchorage being in two parts, each part anchoring the six cables on its side of the bridge. The upward pull of the cables at each anchorage (one side) is 54,500,000 pounds and the horizontal pull 109,000,000 pounds. The estimates have been made on the basis of connecting the cables with the anchor bars outside of the masonry of the anchorage, placing these anchor bars in tunnels, and connecting them with bearing plates at the lower end; everything would be accessible for care and repairs. The chains would be of steel eyebars which have been proportioned for a stress of 20,000 pounds per square inch with an allowance of 20 per cent for details. The estimated weight of the bars and pins in each of the four half anchorages is 6,825,000 pounds, while the plates at the bottom would add 600,000 pounds to this amount, making the total weight in each half anchorage 7,425,000 pounds, or 29,700,000 pounds in the four.

Structural steel.—In estimating the cost of the structural steel work, your Board have used the same price per pound as for the work in the cantilever bridge, namely, 4½ cents. On this basis the cost would be as follows:

	Pounds.
Suspended superstructure	100, 921, 600
Towers	76, 047, 000
Chains	27, 300, 000
Anchor plates	2, 400, 000
Structural steel	206, 668, 600, at 4½ cents, $9,300,087.

The majority of the Board believe that this price is too high, owing to the difference in character of steel work in the two structures, and that the total cost of the structural steel work should not be estimated higher than $8,500,000.

Wirework.—The cables and suspenders have been estimated at 8 cents per pound. making their cost—

	Pounds.
Cables	79, 647, 800
Suspenders	4, 560, 000
Total wire	84, 207, 800, at 8 cents, $6,736,624.

Superstructure.—The total cost of the superstructure is $16,036,711, on the basis of 4½ cents for all structural steel.

Substructure.—The substructure would consist of two anchorages and the bases for two towers. Each tower base has to carry the following weights:

	Tons.
Suspended weight on top of tower	109, 000
Tower	19, 000
Extra effect of wind	4, 000
Total	132, 000

Each of the tower bases of the 2,000-foot cantilever bridge carries 100,000 tons. In both cases the foundations can be made proportional to the weights carried.

The east tower is in the same place as the east pier of the cantilever bridge. The cost of this base for the suspension-bridge tower will be that of the cantilever-bridge pier, or $3,464,000, multiplied by 1.32, making $4,572,480.

The west tower would come immediately west of the New Jersey pier-head line, the average depth of rock being about 10 feet more than on the east side, requiring 414,000 cubic feet additional in the foundation. Estimating on the same basis as for the west pier of the cantilever bridge, the cost of this 414,000 cubic feet of foundation would be $431,000, which would make the cost of the west tower base $5,003,480.

The anchorages have been planned on the basis of putting the entire weight which is to resist the pull of the cables above mean high water, and the quantities have been based on a coefficient of friction of 0.6 and a factor of safety of 2. The anchorage at each end of the bridge would contain 5,940,000 cubic feet above the foundation. The only duty of the anchorages is to act as weight, and a very cheap class of masonry can be used for this purpose; rubble made of the most available stone, with a facing of rough ashlar or brick, would do. The cost of this masonry has been estimated at 37½ cents per cubic foot, although the Board believe it could be built for much less. On this basis the cost of each anchorage above mean high water is $2,227,500.

The east anchorage would be founded where the rock is 20 feet below mean high water; the foundation could be put in with an open cofferdam, and has been estimated as costing 75 cents per cubic foot. There would be 1,150,000 cubic feet in this foundation, making the cost $862,500, and the total cost of the east anchorage $3,090,000.

The foundation of the west anchorage would have to be sunk 60 feet to reach the rock, and would probably be put in by the pneumatic process. Its volume would be three times that of the east anchorage, and its cost may be estimated at the same price per cubic foot, or $2,587,500, making the total cost of the west anchorage $4,815,000.

The total cost of the substructure would then be:

East anchorage	$3,090,000
Base for east tower	4,572,480
Base for west tower	5,003,480
West anchorage	4,815,000
Substructure	17,480,960

The anchorages can be adapted to carry the tracks, but the tracks must be carried between them and the towers on viaducts, requiring 925 feet of viaduct on each side, or 1,850 feet in all, which has been estimated at the same price as before. The total length of the suspension bridge, including viaducts and anchorages, is 5,600 feet. The total cost will be as follows:

Superstructure	$16,036,711
Substructure	17,480,960
	33,517,671
Viaduct	1,850,000
Total	35,367,671

The estimated cost of the 2,000-foot cantilever bridge was $25,443,000 for 4,320 feet; to compare it with the 5,600-foot suspension bridge, 1,280 feet of viaduct must be added; this makes the cost $26,723,000; the estimated cost of the suspension bridge is $8,644,671 more. The fairest comparison is by percentages; the cost of the suspension bridge is nearly 32½ per cent more than that of the 2,000-foot cantilever bridge. If allowance is made for cost of structural steel in accordance with the views of a majority of the Board, the difference will be reduced to $7,844,584, or nearly 30 per cent. The general conclusion which your Board have reached is that the cost of a suspension bridge of a single span, designed for its whole length for the same moving load as the 2,000-foot cantilever bridge, would be less than one-third more than that of the cantilever.

Deflections.—The structure described is one of unusual rigidity. The expansion of the metallic towers counteracts in a degree deflections due to elongation of cables under an increase of temperature, this deflection being further reduced by the large versed sine. Of the 34 tons per linear foot, only 9 are moving load, so that the stress per square inch on cables caused by a maximum moving load is less than 16,000 pounds; it would not exceed 5,000 pounds with an ordinary freight train on every track, or 2,500 pounds with a passenger train on every track. The deflections have been calculated for a full moving load with the following results:

Conditions.	Effects of— Cable between towers.	Back stays.	Towers.	Suspenders.	Total.
60 degrees ± F.	∓2	∓0.1	±0.22	∓0.03	∓1.91
Maximum mov. load	−2.75	−0.14	−0.11	−0.02	−3.02
Combined	−4.75	−0.24	+0.11	−0.05	−4.93

In other words, the total deflection at the center of the span below a mean is about 5 feet; the deflection above a mean is less than 2 feet; the total range is less than 7. These deflections are within satisfactory limits for railroad service.

A deflection of 5 feet in a length of 3,200 feet, calculated for a modulus of elasticity of 28,000,000 pounds, corresponds to a chord stress of

7,870 pounds per square inch in a stiffening truss 120 feet deep. This is the stress which has been eliminated by the use of the hinge.

LIGHTER STRUCTURE.

The calculations of the cost of the suspension bridge which has been described have been made as nearly as possible on the same basis as the estimates for the cantilever bridge, without taking into consideration the fact that the cantilever bridge would be strained nearly to its full capacity by a load 1,000 feet long, while the suspension bridge would be fully strained only when covered by a load three times that length. Furthermore, no allowance has been made for the fact that the maximum strains in the stiffening truss would occur only under combinations which might not arise once in a century, and which could be prevented by simple police regulations.

A moving load of 3,000 pounds per foot, 1,000 feet long, on each of the 6 tracks, crossing the bridge without change of relative position, would produce practically maximum effects in upward moments on a continuous stiffening truss, but it would produce only one-half these moments downward. In other words, the chords of the stiffening truss would be strained 12,500 pounds per square inch by upward bending, but only 6,250 pounds by downward bending, on the assumption as before that all the moving load is distributed by the stiffening truss; as only about 88 per cent is distributed by reason of the unsymmetrical deflection of the cable, the maximum chord stresses are reduced and become, respectively, 11,000 and 5,500 pounds. The greatest upward deflection from the action of the cables occurs from the effects of temperature when the bridge is unloaded; under a full load it is eliminated, and under a 1,000-foot load it is reduced to about 1 foot, which corresponds to a chord stress of 1,570 pounds, making a total of 12,570. The downward deflection would never exceed 3½ feet with the limited length of train, which corresponds to a chord strain of 5,509 pounds, or a total of 11,099, so that a continuous truss could be used without exceeding the assumed limits of stress. It should be noted that the only condition which would produce these stresses would be the passage of six maximum trains side by side. A single freight train in the most unfavorable position would produce a stress of not over 3,500 pounds in the chords of the stiffening truss, and a single passenger train a stress of not over 1,800 pounds. In providing for a lighter structure adapted to trains 1,000 feet long, it has been thought best to make no reduction in the weight of the stiffening truss or the floor system, but the continuous form of truss might be selected.

If the stiffening truss did its complete duty in the distribution of weight, the greatest strain which a train 1,000 feet long, weighing 3,000 pounds per foot, could throw upon the cables, would correspond to a uniform load of 937 pounds. If the stiffening truss did no duty whatever, but the weight was distributed strictly according to the laws of leverage, the greatest strain which such 1,000-foot train could throw upon the cables would correspond to a uniform load of 1,582 pounds per linear foot. Under these circumstances it seems safe, while not reducing the stiffening truss, to provide for a moving load on the cables of only 1,500 pounds per foot of track. For this approximate calculation, the weights per linear foot may then be taken as follows:

	Pounds.
Suspended superstructure and tracks	34,000
Moving load	9,000
Cables and suspenders	14,000
Total	57,000

This is 28½ tons per linear foot, instead of 34 tons, the reduction in the total carrying capacity being about 16 per cent. It should be observed that the live load is only 15.8 per cent of the whole, so that the additional stress put on the cables by the simultaneous passage of six maximum trains would, without allowance for the work of the stiffening truss, be only 10,000 pounds per square inch. The stress imposed by a 1,000-foot passenger train under the most unfavorable conditions would not be over 1,200 pounds.

For purposes of the present comparison, the suspended superstructure remains unchanged; all other parts may be taken at 16 per cent less than in the previous estimate. The weights and cost of such a bridge may then be estimated as follows:

	Pounds.		
Suspended superstructure	101,000,000		
Towers	64,000,000		
Chains and anchor plates	25,000,000		
Structural steel	190,000,000,	4½ cents	$8,550,000
Wire work			5,659,000
Total superstructure			14,209,000
Substructure			14,684,000
			28,893,000
Add for viaduct			1,850,000
			30,743,000

This is $4,625,000 less than the previous estimate, and $4,020,000, or about 15 per cent, more than the cost of the cantilever with the 2,000-foot clear span.

This estimate has been made for the purpose of comparing on the same basis, that of a factor of safety of three on ultimate strength of metal, the 2,000-foot cantilever and the suspension bridge when carrying train loads 1,000 feet long. If it be thought that the stress of 60,000 pounds per square inch on the wire in the cables is too high, it may be noted that the difference in the cost of wirework in the two suspension bridge estimates is $1,017,624, and if the higher cost is restored it will be equivalent to reducing the stress in wire to about 50,000 pounds per square inch. With this change the cost of the lighter structure becomes $31,671,000, this being $5,038,000, or about 19 per cent, more than that of the 2,000-foot cantilever.

If only one train is allowed on one track at a time, maximum stresses will occur in the different members no oftener than the same loads would produce maximum stresses in the 2,000-foot cantilever. Moreover, the load of 1,500 pounds per foot adopted for the cables is the full weight of a passenger train and would not be exceeded if the entire span were covered with the heaviest class of passenger equipment.

UPPER LOCATION.

If a location near Sixty-ninth street were adopted, the conditions would be a little more unfavorable for the foundations of the towers, but very much more favorable for the anchorages, as rock is found above water on both sides. The cost of the two anchorage foundations is $3,450,000 in the first estimate and $2,900,000 in the second estimate; these foundations would be saved at the upper location. These figures are enough to show that there are points within the limits prescribed by the act where the difference in cost between the 2,000-foot cantilever and the single-span suspension bridge might be much less than has been estimated.

CONCLUSION.

The only subject referred to your Board is to "recommend what length of span not less than 2,000 feet would be safe and practicable for a railroad bridge to be constructed over" the Hudson River between Fifty-ninth and Sixty-ninth streets. A single span from pier head to pier head, built on either the cantilever or suspension principle, would be safe. The estimated cost of the 3,100-foot clear-span cantilever being about twice that of the shorter span, your Board consider themselves justified in pronouncing it impracticable on financial grounds. As the cost of the single-span suspension bridge is at most one-third greater than that of the 2,000-foot cantilever, your Board are unable to say that such greater cost is enough to render the suspension bridge impracticable.

The Board have reached this conclusion after careful study, and they have thought it best to give the full course of reasoning which they have followed. They feel that the contingency attending the construction of the deep river foundation of the cantilever bridge, even waiving the absolute necessity of carrying this foundation to rock, is enough to balance a part of the greater cost of the suspension bridge.

The conclusion of this Board is that of a Board of Bridge Engineers acting in a professional capacity. While from such professional view they must pronounce the suspension bridge practicable, they do not in this conclusion give an opinion on the financial practicability and merit of either plan.

Before closing, your Board desire to state particularly that the estimates have been made for comparative purposes and are not to be taken as a measure of absolute cost; they are believed to be thoroughly fair for comparisons; the prices assumed may be much higher than absolute cost. The plans on which the estimates are made, a sketch of which accompanies this report, would undoubtedly be modified if a bridge were built.

This report is accompanied by the following appendices:

A.—Act approved June 7, 1894.
B.—Statement prepared by Mr. Charles Macdonald in behalf of the New York and New Jersey Bridge companies. (Six inclosures, including four blue prints.)
C.—Statements of Mr. Gustav H. Schwab (C) and Wm. W. Hildenbrand (C^1 — C^2). (Two inclosures, tracings.)
D.—Statement of Mr. G. Lindenthal.
E.—Theoretical investigation of stiffening truss.

Respectfully submitted.

G. BOUSCAREN.
W. H. BURR.
THEODORE COOPER.
GEO. S. MORISON.
C. W. RAYMOND.

Hon. DANIEL S. LAMONT,
Secretary of War.

INDORSEMENT OF SECRETARY OF WAR ON THE FOREGOING REPORT.

WAR DEPARTMENT,
December 12, 1894.

The Board having reported that a single span from pier-head to pier-head would be safe and not impracticable, I approve such report and plans may be submitted for a bridge with a single span from pier-head to pier-head.

DANIEL S. LAMONT,
Secretary of War.

APPENDIX A.

ACT APPROVED JUNE 7, 1894.

[PUBLIC—No. 83.]

AN ACT To authorize the New York and New Jersey Bridge Companies to construct and maintain a bridge across the Hudson River between New York City and the State of New Jersey.

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That the New York and New Jersey Bridge Companies, heretofore incorporated by the States of New York and New Jersey, and existing under the laws of said States, are hereby authorized to construct, operate, maintain, and rebuild, in case of destruction, a bridge across the Hudson River between New York City, in the county and State of New York, and the State of New Jersey, subject to the laws of said States, respectively, upon the following terms, limitations, and conditions:

First. That the location of said bridge shall be subject to approval by the Secretary of War, upon such examinations, hearings, and reports as he shall hereafter prescribe: *Provided,* That it shall not be located below Fifty-ninth street, New York City, nor above Sixty-ninth street, New York City.

Second. That the said companies may locate, construct, and maintain over such bridge and the approaches thereto railroad tracks for the use of railroads: *Provided,* That any railroad on either side of said river shall be permitted to connect its tracks with the said bridge approaches, and shall have equal rights of transit for its rolling stock, cars, passengers, and freight upon equal and equitable terms, and if a dispute as to the equality or equity of the terms shall arise it shall be submitted to and decided by the Secretary of War: *Provided,* That the location of all approaches of said bridge in the city of New York shall be approved by the commissioners of the sinking fund of the city of New York: *And provided further,* That no railroad or railroads shall be operated on the approaches of said bridge companies in the city of New York, except on such approaches as shall have been approved by the sinking-fund commissioners of the city of New York: *Provided, also,* That the term approaches as used in this Act shall be construed to include only such portion of the roadbed and superstructure, on either side of said bridge, as is necessary to reach the grade of the bridge from the grade of the streets at which said approaches begin to rise, in order to bring the two elevations together upon and by a grade of not less than twenty feet to the mile.

Third. That any bridge built under the authority of this Act shall be constructed with such length of span and at such elevation as the Secretary of War shall approve and require: *Provided, however,* That it shall afford, under any conditions of load or temperature, a minimum clear headway above high water of spring tides of not less than one hundred and fifty feet at the center of the span; and all the plans and specifications, with the necessary drawings of said bridge, shall be submitted to the Secretary of War for his approval, and before such approval the construction shall not be begun; and should any change be made in said plans during progress of construction, such changed plans shall be submitted to said Secretary and approved by him before made; and the President shall appoint a board, consisting of five competent, disinterested, expert bridge engineers, of whom one shall be either the Chief of Engineers or any member of the Corps of Engineers of the United States Army, and the others from civil life, who shall, within thirty days after their appointment, meet together and, after examination of the question, shall, within sixty days after their first meeting, recommend what length of span, not less than two thousand feet, would be safe and practicable for a railroad bridge to be constructed over said river, and file such recommendation with the Secretary of War, but it shall not be final or conclusive until it has received his written approval. In case any vacancy shall occur in said board, the President shall fill the same. The compensation and expenses of said board of engineers shall be fixed by the Secretary of War and paid by the said bridge companies, which said companies shall deposit with the Secretary of War such sum of money as he may designate and require for such purpose: *Provided, always,* That nothing herein contained shall be construed as preventing the said board of engineers from meeting, investigating, and filing their recommendation after the expiration of said time herein mentioned.

Fourth. The companies operating under this law shall maintain on the bridge, at their own expense, from sunset to sunrise, such lights and signals as the United States Light-House Board may prescribe.

Fifth. The said company or companies availing themselves of the privileges of this Act shall not charge a higher rate of toll than authorized by the laws of the State of New York or New Jersey, and the mails and troops of the United States shall be transported free of charge over said bridge.

Sixth. That said company or companies shall be subject to the interstate-commerce law, and to all amendments thereof, and when such bridge is constructed under the provisions of this Act it shall be a lawful military and post road and a lawful structure.

Seventh. That the said company or companies availing themselves of the privileges of this Act shall file an acceptance of its terms with the Secretary of War, and shall submit to the Secretary of War, within one year after the passage of this Act, for examination and approval, drawings showing plan and location of the bridge and its approaches; and the construction of said bridge shall be commenced within one year after said location and plans have been approved of, as herein provided; and said company or companies shall expend, within the first year after construction has commenced, as herein required, not less than two hundred and fifty thousand dollars in money, and in each year thereafter not less than one million of dollars in money in the actual construction work of said bridge, which shall be reported to the Secretary of War; and the said bridge shall be completed within ten years from the commencement of the construction of the same, as herein required; and, unless the actual construction of said bridge shall be commenced, proceeded with, and completed within the time and according to the provisions herein provided, this Act shall be null and void.

The right to amend, alter, modify, or repeal this Act is hereby reserved.

Approved, June 7, 1894.

Appendix B.

STATEMENT OF MR. CHARLES MACDONALD, OF UNION BRIDGE COMPANY.

No. 1 Broadway, New York, *July 20, 1894.*

Sir: In accordance with your permission, I have the honor to submit herewith certain general information relating to the proposed bridge across the Hudson River, authorized by recent act of Congress, and under which act your Board has also been constituted.

The bridge in question must be located between Fifty-ninth and Sixty-ninth streets and your Board is to advise as to what is a safe and practicable span, not less than 2,000 feet, for such a location.

The New York and New Jersey Bridge Company has in view the construction of a bridge for utilitarian purposes only. There is to be nothing of the monumental or sentimental character about it, except in so far as must be inseparably connected with its magnitude. It is intended to build a structure which will safely provide facilities for all the traffic which may be expected to pass over it, as well as under it, and at such practicable cost as will prove attractive to the investors of capital. It is not committed to any particular design of bridge, whether it be cantilever, suspension, or a combination of the two. What it intends to build, if permitted to do so, is a structure which will accomplish the desired results with the least expenditure of money.

The first element in this problem is, necessarily, the determination of the probable traffic. From a careful observation of the number of cars arriving at Jersey City, the amount of express freight, and the number of passengers which would be likely to pass over the bridge, it has found that the gross income (from all sources) will not exceed $3,500,000, that the cost of maintenance, taxes, etc., would be $1,250,000, leaving as available for payment of interest, after deducting all other charges, $2,250,000. This, at 5 per cent, represents a total investment of $45,000,000. Of this amount about one-half will be required for terminals (including right of way).

Thus it would appear that if the bridge proper can be built at a cost of $22,500,000 the entire investment might possibly be favorably considered. Our efforts have, therefore, been confined within this limit of cost, and the results are respectfully presented for your consideration.

It is proposed to build a "cantilever" bridge, having a span of 2,300 feet between centers of main towers, or upward of 2,000 feet in the clear, and 2 anchor spans of 910 feet each between center of main tower and anchorage pier.

The clear height above high water, at center of main span, will be 150 feet.

There will be 6 railway tracks throughout the entire length of bridge and approaches.

The main towers will rest upon 4 cylinders, each arranged in the form of a square, of 200 feet between centers. They will finish off with granite masonry at a height of 25 feet above high water.

The anchorage piers will be founded upon cylinders and finished off with granite masonry to the underside of the bottom chord of the trusses.

In accordance with the law of the State of New York upon which the charter is based, the main tower on the New York side is placed wholly within the pierhead line. The anchorage pier on the New Jersey side is placed wholly within the pierhead line on that side of the river, and the center of the river pier is about 900 feet eastward from this same line.

Accurate borings have been made at different points within the limits of location permitted by the act, from which it appears that the distance from high water to rock or bowlder (at the site of the river pier) is upward of 250 feet, while sand is found at a depth of 165 feet.

It is proposed to construct this river pier on a sand foundation, at a depth of 200 feet below low water. The diagram submitted herewith indicates the general dimensions and pressures for each of the 4 cylinders composing this foundation.

The cylinders under the New York pier would be of the same dimensions, but the depth to a suitable sand foundation would be somewhat less.

Detailed strain sheets are herewith submitted, showing the general distribution of material required to insure safety for the live load on six tracks having an average of 3,000 pounds per running foot on each track.

Suitable provision has also been made for wind strains and for strains during erection wherever they exceed normal strains.

It will be observed, by reference to the diagram of the foundation cylinder, that the total pressure (from live and dead loads and wind strains) upon the top of the granite capping is 8.84 tons per square foot, and that the abnormal pressure on the base, where the concrete filling comes into contact with the sand (at a depth of 200 feet), is 7.16 tons per square foot.

The nearest precedent believed to be in existence for a deep foundation of this character is the pier foundation for the Hawkesbury bridge in New South Wales, a diagram of which is herewith submitted. The abnormal pressure per square foot in this latter case is 5.7 tons, with a depth of only 8 feet in the sand, and at a total depth of 162 feet below high water.

As it is well known that the resisting force increases with the depth, it is believed that the assumption herein taken is justifiable, but in order to make sure it is proposed (and provision has been made in the estimate for same) to sink a trial cylinder, 20 feet diameter, in the center of the square between the four cylinders composing the river pier. From the experimental data thus obtained as to the exact amount of skin friction and resistance to settlement, more accurate proportions can be given to the main cylinders, particularly with reference to the relation of weight required to cause settlement during dredging.

It will be observed that the effect of skin friction has not been considered in calculating the supporting value of the foundations. This will be wholly on the side of safety, therefore, and will unquestionably reduce the abnormal pressure on the sand at the foot of the cylinder.

These cylinders will be filled up with concrete made of the best Portland cement, lowered through the water in the most approved manner, and finished off at about the level of the bottom of the river. The outer skin of the cylinder will be carried up above high water, temporarily, to facilitate the construction of the masonry from the river bottom upward.

In the completed pier there will be no metal work exposed to corrosion where it might give rise to anxiety.

It is proper to state that what is called "granite masonry" consists of a 4 feet ring of cut granite and 4 feet of dressed granite coping; the interior to be made up of large irregular masses of stone, set in concrete, exactly as was done in the case of the piers for the "Forth Bridge."

The estimate of cost of the river pier and the New York pier is herewith given in detail, together with the cost of superstructure, and other items to make up the total cost of the bridge proper, from which it will appear that, with a span of 2,300 feet between centers of towers (or a clear span of upward of 2,000 feet), the limit of what has been found to be a practicable expenditure is reached.

In view of the evidence presented, you are respectfully requested to consider favorably the following propositions:

(1) That a 2,000 feet clear span is the longest practicable span wherewith to cross the North River, at the point indicated.

(2) That any increase of span, short of the entire width of the river, viz, 3,130 feet in the clear, would correspondingly restrict the free use of wharves on the New Jersey side.

(3) That the cost of a river pier, at any point between the present location and say 500 feet from the New Jersey side, would be practically the same as already estimated; whereas, the cost of the superstructure would increase materially with the length of the span.

(4) That the position of the river pier, assumed at 2,000 feet in the clear from the New York side, would be wholly within the space set apart as anchorage grounds on

the New Jersey side; as indicated by diagram attached hereto (taken from the New York Times of Thursday, September 7, 1893); from which it will be seen that 1,500 feet was assumed to be sufficient for the free and unobstructed navigation of the Hudson at that point.

And further, that such pier, when provided with suitable warning signals, would be a positive advantage to navigation in time of fog.

And further, that the piers of the Poughkeepsie bridge, crossing the Hudson River 75 miles above, are 500 feet apart in the clear, and have not proved a serious obstruction.

And further, that the main ship channel to New York Harbor is 1,000 feet wide.

(5) That the superstructure of a span without a pier in the river, for which it would be necessary to have a clear reach between centers of towers of 3,350 feet. would cost at least three times as much as the estimate herewith submitted; and the total cost of the bridge proper would be more than double the estimate for a 2,000 feet span.

We have no hesitation in saying that, at such an increase of cost, it would be impossible to raise the necessary capital; and that, therefore, the bridge would be impracticable.

In arriving at this conclusion we have been guided by the carefully digested opinions of men of large means, who have expressed a firm belief in the enterprise, if kept within the prescribed limit of cost; and from whom we should expect substantial assistance in perfecting a sound financial basis of operations.

I do not wish to be understood as admitting that a bridge of 3,350 feet can be constructed as a safe structure.

We have made some preliminary estimates of strains for such a span, and the proportions have become so enormous as to raise very grave doubts in our minds of the possibility of designing such a structure, with sufficient rigidity to hold up its own weight—to pass the traffic—and to resist wind pressure.

All of which, and such other data as may be in my power to procure, is placed at your service.

In inclosure No. 1 will be found data upon which the estimate of probable traffic is based.

In inclosure No. 2 references are given to loads upon masonry and foundations as they have been hurriedly collected.

The following plans accompany this report:

Five copies, pier foundation.

Five copies, plan of river, with borings.

Ten copies, profile at proposed crossings.

Three copies, strain-sheets of superstructure.

One copy, article from the New York Times, September 7, 1893.

CHARLES MACDONALD,
Of Union Bridge Company,
No. 1 Broadway, New York.

Maj. C. W. RAYMOND,
Chairman Board of Engineers appointed by the President upon the matter of length of span of the New York and New Jersey bridge over the Hudson River.

NOTES AS TO PRESSURES ON MASONRY AND FOUNDATIONS.

[Collingwood, "Masonry East River Bridge," Transactions Am. Soc. C. E., Vol. VI, pp. 8 and 9.]

Weight per cubic foot: Granite masonry, 153 pounds; concrete, 120 pounds. Pressure on central shaft, 26 tons per square foot.

[Cresy, "Encyclopedia of Engineering, 1847."]

(Page 705.) Five and one-half tons on 9 square inches, 1,370 pounds per square inch, has stood for several centuries. "Chapter House at Elgin." Two columns in the Church Toussant d'Augers, 12 inches diameter and height 25 feet, carrying pointed arches; load on each, 25 tons, or 400 pounds per square inch.

(Page 706.) Piers: Dome St. Peter's, 1,022½ pounds on 9 square inches, or 113 pounds per square inch; St. Paul's, 1,190 pounds on 9 square inches, or 132 pounds per square inch; Invalides, 922 pounds on 9 square inches, or 102 pounds per square inch; St. Genevieve, 1,840 pounds on 9 square inches, or 204 pounds per square inch. Columns: St. Paul's, without the walls, 1,235 pounds on 9 square inches, or 137 pounds per square inch; Church Toussant d'Augers, 2,767 pounds on 9 square inches, or 307 pounds per square inch.

The above is quoted from "Rondelet, Traité d'Architecture."

[John Newman, "Cylinder Bridge Piers," approximate safe loads per square foot.]

Firm sand in estuaries and bays, 5 to 5.6 tons. Dutch engineers consider safe loads on firm, clean sand 6 to 6.16 tons. Very firm, compact sand—foundations at considerable depth, not less than 20 feet—and sandy gravel, 6.7 to 7.84 tons. Firm shale and clean gravel, 6.7 to 8.96 tons. Compact gravel, 7.84 to 10.08 tons.

Clean sand, homogeneous Thames gravel, has been weighted with 280 cwt. per square foot at 3 to 5 feet below the surface, and showed no signs of failure, 15.68 tons.

[Gaudard, "Foundations."]

Stiff clay, marl, sand, or gravel, 55 to 110 cwt. (3.08 to 6.16 tons). Gorai bridge (close sand), Lock Kew (gravel), Bordeaux (gravel), 165 to 183 cwt. Nantes (sand), 152 cwt., some settlement. Szegedin (clay and fine sand), 133 cwt. (7.4 tons), reinforced by driving piles in interior of cylinder, and sheathing outside. Charing Cross, cwt., 159, including adhesion (8.9 tons). Cannon street, 117 cwt., including adhesion (6.5 tons). Roque Favor aqueduct, 258 cwt., rocky ground (14.4 tons).

[Leslie, "Transactions Institute of Civil Engineers, January 24, 1888."]

Hooghly Jubilee bridge, 10 tons net.

[Engineering News, March 14, 1885.]

Washington monument: Area of base, 126.5 feet by 126.5 feet, 16,000 square feet. Weight, 81,120 (long tons), 90,850 (short tons). Average pressure on base (exclusive of wind), 11,340 pounds, or 5.67 tons per square foot. Taking out area 45 feet square, 2,025 feet, leaves balance under concrete, 14,000 square feet (nearly). Average per square foot under concrete, 90,850 ÷ 14,000, 6.5 tons.

	Square feet.
Area of bottom of buttress, 101.5 square	10,302
Less 45 square	2,025
Effective area	8,277
	Tons.
14,000 by 13 feet, 182,000 by 150	13,650
Original weight	90,850
Less	13,650
	77,200

On line c — c, pressure 77,200 ÷ 8,277, 9.32 tons per square foot.

Area at c — c, 55 square, 3,025 — 25 square, 2,400 square feet.

Weight of buttress, 101.5 square	10,302
55 square	3,025
4 by 78 square	24,336
	37,663 ÷ 6, 6,277 by 150 by 25 = 23,540,000 pounds, 11,770 tons.

Weight of shaft, 77,200 — 11,770, 65,430 tons.

Average per square foot, 65,430 ÷ 2,400 square feet, 27.2 tons per square foot.

Material is marble.

At bottom of foundations, 5.67 tons per square foot; at bottom of buttress, 9.32 tons per square foot; at bottom of shaft, 27.2 tons per square foot; all exclusive of wind.

Bunker Hill Monument: On hard sand and gravel, 5½ tons, no settlement.

Tower of brick church (Thirty-seventh street and Fifth avenue): On hardpan, 7 tons per square foot, some settlement.

Crushing strength of concrete, department of docks: 1 : 2 : 5 (1 foot cube). Hardened in water forty-five days, 425 pounds square inch, 30.5 tons square foot. Hardened in water one year, 1,520 pounds square inch. Hardened in air one year, 1,620 pounds square inch.

REVENUE STATISTICS.

I regret to say that much of the detailed information which I expected to submit in this appendix has been forwarded to London, but the following will be of interest:

North River ferries, passengers carried yearly.

Staten Island	5,445,800
Jersey Central	10,938,320
Pennsylvania	14,589,050
Barclay	12,899,100
Chambers	10,868,240
Jay	2,164,640
Desbrosses	8,067,960
Christopher	11,739,130
Fourteenth street	2,811,960
Twenty-third street	3,594,520
Forty-second street	1,744,680
Total	84,663,400

The number of people crossing the East River ferries is slightly in excess of the above, or more exactly 88,663,500; but this is exclusive of the people who cross the Brooklyn bridge.

These latter amounted to 42,615,105 in 1893.

From the opening of the railway to public use, September 24, 1883, to November 30, 1893, inclusive, a period of ten years and sixty-seven days, 304,875,286 passengers were carried. During any month the greatest number transported was 4,033,920 in October, 1892, which included the week from the 8th to the 15th of the Columbian festival. The next greatest number was 3,846,493, in May, 1893, an average of 124,080 per day. In one day of twenty-four hours, the maximum number carried was 223,625, on October 12, 1892, during the Columbian festival; the next greatest number was 166,403, on January, 14; and the minimum number during the official year 1893 was 45,280, on August 20.

In 1884, the year after the opening, the total number of passengers passed over the bridge was 8,828,200.

The average number of cars of freight, inbound and outbound daily, on the following-named railroads, via New York and Jersey City, during the year 1890, as far as statistics have been obtained, is as follows:

	Pennsylvania Railroad.	Erie.	West Shore.	Delaware, Lackawanna and Western.	Jersey Central.	Various.
N. and W., through	318	322	130	144	138	48
E. and S. and way	724	694	260	402	328	244
Estimated coal	300	210	....	210	200	....
	1,342	1,226	390	756	666	292

Grand total, 4,672 cars daily; 1,705,280 cars during year.

Among the above items of freight which could be handled to advantage in New York may be mentioned—

Milk	daily carloads	53
Flour and meal	do	397
Produce	do	407

Grand Central Station.—Total passengers per day, 35,000; or 12,250,000 per year.

St. Louis bridge.

	1890.	1891.	1892.	1893.
Loaded freight cars	259,187	224,784	232,259	214.816
Empty freight cars	178,197	132,187	141,062	139,023
Passenger coaches	111,350	115,942	125,676	128,601
Baggage, mail, and express	46,775	50,009	50,696	51,568
Construction cars	12,948	609	4,847	7,605
Revenue per loaded car	\$4.50	\$4.34	\$4.46½	\$4.58
Average number of cars per day	1,940	1,434	1,515	1,484
Number of passengers	1,367,184	1,375,057	1,522,037	1,587,549
Revenue per passenger .... cents..	25.84	26.15	21.7	24.6
Average number passengers per day	4,149	4,231	4,642	4,848
Ratio of expenses to earnings .... per cent..	41.77	46.57	44.94	43.31

It has been assumed that the suburban traffic which can be brought into the terminal of the North River bridge, at Forty-second street and Seventh avenue, will equal, if it does not exceed, that which is delivered at the Grand Central Station, Forty-second street and Fourth avenue; and that of the western passenger traffic now reaching New York, of which 70 per cent is carried by the New York Central system, against 30 per cent by all other lines, a very considerable diversion will take place in favor of the new terminal.

Based upon this, the total number of passengers paying toll over the bridge has been taken at 14,000,000 per year. Of quick freight, including express, it is safe to count upon 1,000 cars per day; or, say, 350,000 per year.

In assuming a rate which it would be safe to charge, per passenger and per car, the cost of motive power has been eliminated; that is to say, the cost of moving the passengers and cars would be borne by the railroads transporting them.

Under this assumption, an average of 15 cents per passenger and $4 per car gives a gross income as follows:

14,000,000 passengers, at 15 cents	$2,100,000
350,000 cars, at $4	1,400,000
Gross income	3,500,000
Less repairs, taxes, and sundries (about 36 per cent)	1,250,000
Net income	2,250,000

By reference to the St. Louis bridge charges for freight and passengers, which include cost of motive power, these rates are moderate.

Estimated cost of New York and New Jersey bridge.

Superstructure, 230,000,000 pounds, at 4½ cents	$10,350,000
River pier	3,500,000
New York pier	2,300,000
New Jersey anchorage	400,000
New York anchorage	100,000
Tracks, 4,000 linear feet, at $2.50 × 6	60,000
Interest	2,000,000
Contingencies	1,890,000
	20,000,000
Add 10 per cent	2,000,000
Total	22,000,000

Estimated cost of river pier, on basis of 60 feet diameter on top and 100 feet diameter on bottom, with inclined sides.

Excavation, 1,178,100 cubic feet, at 27½ cents	$324,000
Concrete, 38,733 cubic yards, at $6	232,400
Steel, 3,000,000 pounds, at 3 cents	90,000
Masonry, 8,600 cubic yards, at $15	129,000
Cost of one cylinder	775,400
Cost of pier, $775,400 × 4 cylinders	3,101,600
Add 10 per cent	310,000
	3,411,000
Test pier	89,000
Total cost of river pier, say	3,500,000

Detailed weight of New York and New Jersey bridge.

	Pounds.
Suspended span, 720 feet c to c end pins:	
Top chords	2,400,000
End posts	1,900,000
Bottom chords	2,900,000
Web eyebars	1,500,000
Vertical posts and braces	800,000
Pins	200,000
Lateral system	700,000
Stringers	2,500,000
Cross floor beams	1,600,000
	14,500,000
Two cantilever arms:	
Top chord eyebars	13,400,000
Web eyebars	7,500,000
Bottom chords	15,800,000
Pins	1,000,000
Web compression members	9,000,000
Lateral system	2,800,000
Stringers	5,000,000
Cross floor beams	4,200,000
Sundries	3,300,000
	62,000,000
Two anchorage arms, 840 feet c to c end pins:	
Eyebars	21,200,000
Bottom chords	21,000,000
Web compression members	34,000,000
Pins	2,000,000
Lateral system	3,500,000
Stringers	5,500,000
Cross floor beams	5,000,000
Sundries	3,300,000
	95,500,000
Two center towers:	
Eyebars	6,900,000
Tower vertical posts	24,900,000
Bottom chords	5,900,000
Lateral system	11,600,000
Stringers	1,100,000
Cross floor beams	1,900,000
Bedplates	5,700,000
	58,000,000
Totals:	
Suspended span	14,500,000
Two cantilever arms	62,000,000
Two anchorage arms	95,500,000
Two towers	58,000,000
	230,000,000

APPENDIX C.

STATEMENT OF MR. GUSTAV H. SCHWAB, CHAIRMAN SPECIAL COMMITTEE, CHAMBER OF COMMERCE OF THE STATE OF NEW YORK, ON HUDSON RIVER BRIDGE.

NEW YORK, *July 17, 1894.*

GENTLEMEN: In accordance with your permission I avail myself of your courtesy to present to you the views of the Chamber of Commerce of the State of New York as represented by its special committee on the Hudson River bridge.

The Chamber of Commerce on December 7 last adopted the following resolutions:

"*Resolved*, That in the opinion of this Chamber the passage by Congress of any Bill permitting the construction of a bridge across the Hudson, with piers in the river bed, will be an obstruction to the commerce of this Port and an injury to the entire country, particularly to the great West, whose products find an outlet through the Erie Canal and the Hudson River.

"*Resolved*, That the Representatives in Congress from this State be requested to strenuously oppose the passage of any Act which will permit the building of piers or other obstructions in the river bed."

The objections urged by the chief commercial body of this city against the proposed location of a pier or piers in the Hudson River between the pier-head lines, opposite the city of New York, are the following:

The lower part of the Hudson River not only serves the purposes of river traffic and of the accommodation of the enormous trade that finds its way from the great West through the Erie Canal to tidewater, and from the brick, lumber, and stone yards, manufactories and ice houses along the river, but this great river between the New York side and the New Jersey shore furnishes such a harbor as can not be found in any other part of the world. It provides the most varied kind of traffic—by steamers, ferryboats, schooners, lighters, rafts, barges, large sailing vessels, and yachts—with accommodation, and renders it possible for the largest ocean steamers to safely maneuver throughout its whole extent. Although at the present time the piers accommodating ocean-steamship traffic do not extend much above the proposed site of the New York and New Jersey bridge, it can not be doubted that with the rapid growth of commerce and navigation of this port, the whole shore line within the city limits of New York on the Hudson River will be ultimately taken up in pier accommodations, as well as the opposite shore on the New Jersey side. This appears a safe prediction, in view of the fact that thirty years ago the harbor shipping of New York did not extend beyond Tenth street. At present it has reached Seventieth street, and on the New Jersey side there are now plans in contemplation and partly in execution for the building of piers as high as Eighty-sixth street.

In connection with this extension of harbor traffic it should be borne in mind that ocean steamships tend to grow larger, and that the space required for their maneuvering should therefore also be larger. It is for this reason that the East River is not used for the handling of large transatlantic steamers, but that these steamers find their docks on the North River, which is by far the most important part of the great harbor of New York, the importance of which is shown by the fact that by a recent bill in Congress the port of New York will now include Yonkers. The placing of a pier or piers in the river bed at any point between the pierhead lines will, inevitably, seriously interfere with the maneuvering of these ocean steamers, as well as with harbor traffic in general. The obstruction to harbor navigation and the great danger to life involved in the construction of a pier or piers in the river bed must be patent to anyone who has crossed the Hudson River in foggy, thick, or stormy weather. It is to be feared that the placing of a pier almost in the center of the river will result not only in an obstruction to the passage of ice in the winter months, but will cause the formation of shoals around the abutments of such piers.

The current in the Hudson River opposite the city of New York does not pursue a course parallel with the river banks, but runs diagonally across from shore to shore, thereby causing the greatest difficulty in handling tows and floats in the harbor. The location of a pier in the river would greatly increase the difficulties and dangers of harbor navigation to those tows, and should a tow of canal boats or a steamer laden with passengers on this crowded waterway have the misfortune to come into contact with the abutments of these bridge piers, the serious consequences to life and property can well be imagined.

The argument has been made in favor of the bill that the proposed pier is to be placed in the anchorage grounds, and not in the main channel. If this is so, the argument displays lack of information, for a pier placed in the anchorage grounds would render a large part of such grounds useless. No vessel of any size could with safety anchor within a circle of 2,400 feet in diameter of which the bridge pier would be the center.

We believe that the existence of a natural rocky island, or islands, of the extent of the proposed bridge pier, in the same location in the river would never be tolerated, and that millions of dollars would have been spent long ago to remove such serious obstructions to navigation in the river and harbor, and in view of the expenditures of the Government for the purpose of removing natural obstructions, the deliberate erection of artificial barriers would appear to be the greatest folly.

The reasons against the pier were so convincing upon the legislature of the State of New York that the act giving a charter to the New York and New Jersey Bridge Company insisted upon a single span for this bridge. The company then appealed to Congress for permission to place a pier in the river; the President, however, vetoed the bill, in view of the danger to the commercial and navigation interests of the first

port of this country. As the bridge company represented in Congress that a span of over 2,000 feet was a practical impossibility, your honorable Board was appointed by the President to examine into the question thoroughly, whether a bridge could be built longer than 2,000 feet span, which, in this case, means a span over the entire river, for if built for 2,100 or 2,300 feet it would still leave a dangerous obstruction in the river between the pierhead lines, and would make the New Jersey shore in the vicinity practically useless for dock purposes. This, then, is the question that comes before your honorable Board, namely, whether a bridge can be built over the river with a single span which would be practicable and not prohibitive in cost.

There is no rule by which the practicability as to cost can be determined, but in the view that a bridge with no pier in the river can be built at a cost that is not prohibitive the commercial and navigation interests of this port find themselves supported by Mr. Thomas C. Clarke, the chief engineer of the New York and New Jersey Bridge Company in his report to the company of March 15, 1892, submitted to the U. S. Senate on March 23, 1892. In this report Mr. Thomas C. Clarke, the chief engineer of the New York and New Jersey Bridge Company, states as follows:

"I expect to be able to have plans and estimates of cost of the suspension-cantilever bridge, requiring no pier in the river between the pierhead lines, ready by April 1st. * * *

"I am now prepared to say that if they decide that there shall be no pier in the river, I can build you a bridge on the combined suspension-cantilever plan that shall be strong enough and stiff enough to carry trains at 20 miles an hour, and at a cost that shall not be prohibitory."

These views of Mr. Thomas C. Clarke, chief engineer of the New York and New Jersey Bridge Company, have the indorsement of Mr. W. A. Roebling, who has written to me under date of June 21 last as follows:

"I am not familiar with the proposed designs for this work, but may say in general that a cantilever with two spans of 2,000 feet each is considered feasible, and that a suspension bridge with a single span of 3,000 feet is also feasible, with some increase in cost. Such a span is within the carrying capacity of a commercial quality of steel wire." * * *

Mr. Roebling also states as follows: "I will close by saying that only two years ago the promoters of the New York & New Jersey Bridge Co. had determined to adhere to the suspension principle, in which I was consulted," * * * thus amply corroborating the statements of the chief engineer of the New York and New Jersey Bridge Company.

The Chamber of Commerce of the State of New York has retained Mr. William Hildenbrand, recommended to them by Mr. W. A. Roebling, and one of his most trustworthy assistants for many years, for the purpose of presenting to you technical arguments in favor of a single span over the whole river, with its probable cost, its practicability, and safety.

GUSTAV H. SCHWAB,
Chairman Special Committee on Hudson River bridge,
Chamber of Commerce of the State of New York.

The BOARD OF ENGINEERS ON HUDSON RIVER BRIDGE SPAN.

APPENDIX C[1].

LETTER OF MR. W. HILDENBRAND TO MR. GUSTAV H. SCHWAB.

NEW YORK, *July 12, 1894.*

DEAR SIR: In answer to your question whether it be possible to construct a practical railroad bridge across the Hudson River at or near Sixty-ninth street without a pier between pierhead lines, I do not hesitate to say yes, and beg to submit to you the following data, which will demonstrate by figures that a suspension bridge of over 3,000-foot span is not only an engineering possibility, but also will compare, from a commercial point of view, not unfavorably with a cantilever bridge of 2,100-foot span, as suggested by the New York and New Jersey Bridge Company.

The design, as submitted to you, must be considered as a mere preliminary suggestion which might undergo many changes if the problem be worked out in detail, but the calculations show what can be done, and I am confident to say that the weight of the metal and the cost as given will not be far from correct figures of carefully prepared plans and estimates.

From a profile of the river, shown in a sketch of the proposed cantilever bridge, published in the Scientific American of June 16, it appears that the distance between pierhead lines is about 3,000 feet, consequently the length of a single-span bridge

was assumed to be 3,200 feet from center to center of towers, allowing 200 feet for the width of the latter.

Most engineers will agree that the suspension principle is the only practical solution for a bridge of that length; hence, without trying any other kind of construction, all calculations are based on a design for a suspension bridge consisting of wire cables and stiffened by a truss with three hinges. This is not the most economical construction, but it was chosen for the purpose of showing the practicability and economy of such a design, even under unfavorable conditions, and, on account of all forces and strains being statically determinate, of easy comparison with a cantilever or other structure.

A suspension bridge is never entirely rigid, because the contraction and expansion of the cables under changes of temperature cause the floor to drop or rise to the extent of several feet. An absolute stiffening of the floor against distortion under one-sided loads would, therefore, be a waste of material, hence the stiffening girder for this assumed design was constructed only with such rigidity that the simultaneous depression and rise of the floor under a one-sided load would create no steeper grade than 1½ per cent, or about the same as is caused by the rise and fall in consequence of extreme temperatures. Keeping these conditions in view, the following are the principal features and dimensions of this bridge:

Total length from face to face of anchorage	feet	4,900
Main span from center to center of towers	do	3,200
Width of tower at water line	do	200
Width of tower at floor line, about	do	100

Eastern end span will consist of two independent truss bridges, each of 400-foot span.

Western end span will consist of three independent truss bridges, each of 266-foot span.

Deflections of cable:		
Main span at 55° F	feet	322
Main span at 0°	do	319.18
Main span at 110°	do	324.77
Rise and fall of cable and floor for a variation of 110°	do	5.59
Length of cable in main span at 55° F	do	3,285.26
Length of cable from anchorage to anchorage	do	5,120
Deflection of back cable from a straight line connecting top of tower and face of anchorage:		
For dead load	feet	9.8
For dead and live load	do	7.2
Depression of center span when fully loaded:		
Arising from the elongation of cable of 5,120 feet length	do	4.21
Arising from the change of deflection in back cable	do	.5
Total rise and fall of floor in center of bridge under extremes of temperature and load	feet	10.3
Camber of floor at 55° F	do	8
Grade of floor at 55° F	per cent	1
Grade of floor at 0° F	do	1.35
Grade of floor at 110° F	do	.66
Camber of floor at 110° and floor fully loaded	feet	.62

From these figures it appears that the maximum grade of the floor is less than 1½ per cent, and that the floor, when fully loaded in the warmest weather, will never sink below level.

The height of the towers will be determined by the following figures:

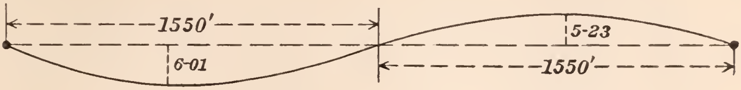
	Feet
From high water to under side of bridge	150
Thickness of floor	8
Camber of floor	8
Bottom of cable above floor in center	2
Deflection of cable	322
Extra height of tower to support two or three tiers of cables	20
Total	510

The bridge is supposed to carry six railroad tracks, and the live load is assumed to be 18,000 pounds per linear foot, covering the whole span from tower to tower.

If the moving load were but 4,500 pounds per linear foot, and one-half the span be covered with the same, it can be shown that, without any stiffening construction, the floor would deflect 6.01 feet at one quarter of the span and rise 5.23 feet at the opposite quarter, making a total difference of 11.24 feet in 1,600 feet, or a grade of 1.4 per cent. This is admissible; hence the stiffening girder must be calculated

to resist a live load of 13,500 pounds per linear foot, and must be dimensioned to deflect not over 6 feet in the quarter span. The span of the stiffening girder is 3,100 feet, and its height was assumed 80 feet, or one-fortieth of the span.

According to the theory of stiffening girders, as shown by Rankine, the loaded half of the truss must be calculated for a uniformly distributed load of $\frac{13500}{2}$ pounds per foot, and the unloaded half for the same force, acting upward, deflecting the girder thus:



Assuming a unit strain of 20,000 pounds per square inch the max. chord section is 1,266 square inches (= 633 square inches for one chord of each of two trusses); the average weight of chords, 7,332 pounds per foot (= 1,833 pounds for one chord of each of two trusses); the average weight of web members, 3,048 pounds per foot; total, 10,380 pounds per linear foot.

Calculating the deflection of this truss, it will be found to be 4.58 feet, if the elongation and contraction of the web members and the separate moment of inertia of each chord be neglected. The true deflection would therefore not exceed 6 feet.

The weight of the platform per double track is about the same as that of any first-class railroad bridge, viz, 1,800 pounds per linear foot.

The floor beams will not reach across the six tracks in one span, but will be supported at two intermediate points; hence the total weight of the platform will be 5,400 pounds per linear foot.

The aggregate load to be sustained by the cables will be composed of the following weights:

	Pounds per linear foot.
Moving load	18,000
Weight of stiffening truss	10,380
Weight of platform, 3,900 pounds steel and 1,500 wood	5,400
Weight of projecting ends of floor beam	300
Weight of sway-bracing and intermediate floor-beam suspenders	810
Weight of wind cables and wind bracing	1,240
Weight of suspenders	1,110
Weight of cables	12,500
Total	49,740

Total load on cable: 12,500 × 3,200 + 37,240 × 3,100 = 77,722 tons; tension in cables at one-tenth deflection = 104,613 tons; allowing a strain of 30 tons per square inch, it requires 3,487 square inches or 74,570 No. 3 wires (diameter, 0.244 inch) which will weigh 12,000 pounds per foot.

Assuming 12 cables, 1 cable will consist of 6,220 wires and will have a diameter, including wrapping, of 23 inches. With 14 cables the diameter of each would be 21¼ inches.

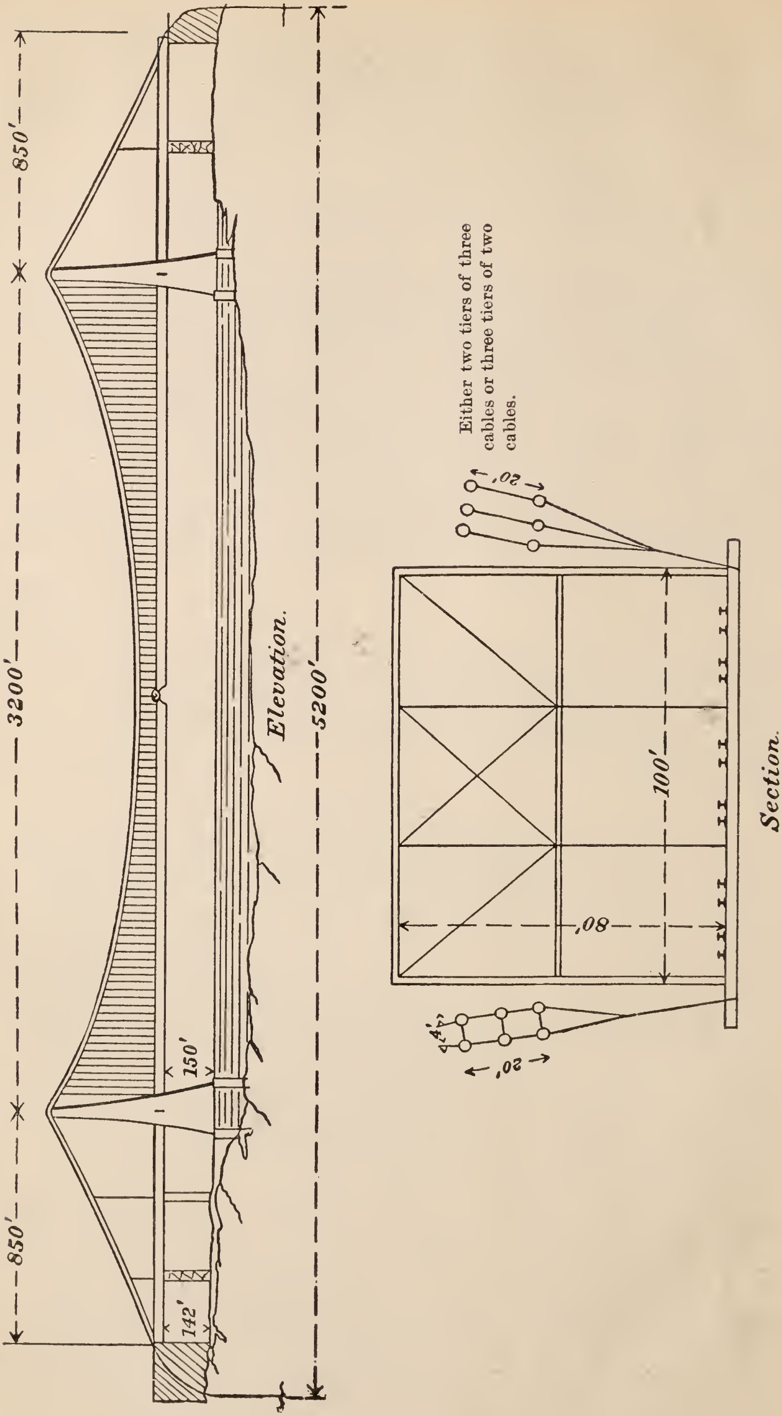
The size and number of cables may be varied according to individual opinion, but the above size may be advisable for coinciding nearest with the cables of the New York and Brooklyn bridge. The latter were designed to contain 6,308 wires, but eventually were built of 5,400 wires of a heavier size.

There is no reason to assume that larger cables could not be made, but there can certainly be no doubt about the successful construction of cables if a size be adopted which is near the limit of the precedent given by the cables of the Brooklyn bridge.

The following sketches will illustrate the general design:

Cable making will require from 16 to 18 months, including the accessory work of erecting "cradle" ropes and foot bridge.

The time consumed for making two of the Brooklyn bridge cables was 9 months, but several methods applied there could be improved upon for shortening the time. For instance, all cable wire was stored on one shore and taken across the river from one side. It required from 7 to 8 minutes for the wire to travel across, while regulating the same took only 2 to 2½ minutes; therefore, the time for strand making can be shortened one-half if the wire wheel, instead of returning empty, would take a wire across from the opposite shore. To "let off" and regulate one strand required from 3 to 4 days' labor, and while this work went on strand making of the Brooklyn bridge cables was interrupted. There is no particular reason for this, and it is fully practical to make a new strand while another is regulated. In this way the time of making cables is actually confined to the time of regulating strands


850'
3200'
850'
150'
142'
Elevation.
5200'
Either two tiers of three cables or three tiers of two cables.
20'
4'
20'
100'
80'
Section.

Each of the 12 cables will consist either of 19 or 37 strands, the former having the advantage of quicker work, but dealing with great weights. The latter makes it easier to handle the strands, though it may take a little longer in regulating them. Assuming 37 strands, and 3 days for regulating each strand, the time of making one cable without wrapping would be 111 days.

Two wrapping machines, working from the towers towards the center, will wrap about 20 feet per day, requiring 160 days for the whole span; but with a sufficient number of "squeezers" it would be just as practical to employ 4 or more wrapping machines and shorten the time accordingly. If the time for wrapping the cable be reduced to 80 days, the total time necessary for making 1 cable will be 191 working or 224 calendar days, which is less than 7½ months. Hence the above-stated time of 16 or 18 months for the whole operation of cable-making gives a liberal allowance for contingencies and for the erection of the auxiliary structures. Of course it will be necessary to make arrangements for constructing all cables simultaneously, which can easily be done, provided the cables are not less than 3 or 4 feet apart.

For attaching the suspenders and making connections between the cables, cable bands may be employed which are made in two parts, provided with heavy flanges, and screwed up with three or more bolts in each flange. The cable bands of the Brooklyn bridge were forged of one piece and screwed up with but one bolt; they were perfectly tight when of correct size, but they had a tendency to slide when a little too large. Bands made in two parts, as described, will never slide if properly constructed, because the number and size of the tightening bolts can be calculated according to the requisite friction.

The unit strain of 60,000 pounds per square inch of cable wire was based on an ultimate strength of 180,000 pounds. Much stronger wire, up to 300,000 pounds, can be made, but that of 180,000 pounds seems preferable on account of its easy manufacture and cheap price. The limit of elasticity of this wire is 120,000 to 130,000 pounds, hence a maximum strain of 60,000 pounds, which rarely if ever occurs, is a perfectly safe assumption.

The stiffening truss is subjected to reverse strains, hence a unit strain of 20,000 pounds per square inch is by many engineers considered equal to one of 40,000 pounds.

While this may be true if the opposite strains occur in rapid succession, namely, at the rate of 5 to 20 and more times per second, as in car axles, it is, on the other hand, known from the experience with rails and continuous bridges that it is not true if the interval between the occurrence of the different strains within the elastic limit affords plenty of time for the recovery of the metal from the elastic deformations. But even if the strains were as high as 40,000 pounds, there could be no objection to it if we adopt high-grade steel, of say 80,000 pounds ultimate strength and 50,000 pounds elastic limit, considering that the maximum strain in the stiffening truss is based on an improbable assumption of load distribution, which may not occur once in a lifetime.

Lastly, it may be mentioned in justification of a high unit strain, that the stiffening truss is not a necessity, but merely a convenience; in other words, it could be dispensed with if a locomotive and train could ascend a 6 or 8 per cent grade; hence a rupture of the truss would not endanger the safety of the suspension bridge, but would merely cause a temporary inconvenience.

The following table gives the calculated weights and the approximate cost of the bridge:

Stiffening trusses, sway braces, and intermediate floor-beam suspenders, 11,190×3,100 = 17,345 tons at 4 cents	$1,387,600
Floor construction, 4,200×3,200 = 6,720 tons at 3¼ cents	436,800
Towers, 26,330 tons at 4¼ cents	2,238,000
Anchor chain and plates, 12,000 tons at 3½ cents	840,000
Woodwork and track, 5,200 pounds at $24	124,800
Cables, 12,000×5,120 = 30,720 tons at 7 cents	4,300,000
Wind cables and suspenders, 2,350×3,200 = 3,760 tons at 8 cents	601,600
East land span, consisting of two 400-foot span truss bridges, weighing per foot, including floor, 13,950 pounds×800 = 5,580 tons at 4 cents	446,400
One land pier, 146 feet high, 416 tons at 4 cents	33,300
West land span, consisting of three 266-foot span truss bridges, weighing per foot, including floor, 9,000 pounds×800 = 3,600 tons at 4 cents	288,000
Two land piers, 146 feet high, 468 tons at 4 cents	38,000
Anchorages	2,500,000
Total, 106,959 tons	13,234,500

This is without the cost of foundations. In explanation of the cost of the anchorges, it may be said that the cables were assumed to be anchored in rock for a depth of 90 feet, and that the masonry above the rock would rise 142 feet above high

water. For each anchorage a 15 by 20 foot shaft is supposed to be sunk in the rock, widened at the bottom to 50 by 50, requiring about 3,500 cubic yards of excavation and subsequent filling with concrete. The resistance of the rock body surrounding two of these shafts is at least 72,000 tons, and if a block of masonry containing 74,000 yards be added to it the total resistance will be 220,000 tons, against a maximum pull of 104,600 tons in the cables. Assuming the cost of excavation at $3 per cubic yard, filling with concrete at $6, and masonry at $14, the total cost of both anchorages will be $2,198,000, or $2,500,000, allowing 15 per cent for contingencies.

The cost of making the Brooklyn bridge cables was 2.05 cents per pound. Wire of the described quality can be bought at 4 or, at the outside, 4½ cents; hence, a price of 6 to 6½ cents per pound for the finished cable would be about correct, while 7 cents was assumed in the estimate of cost.

The estimate must be considered as a liberal one in all items.

It has been mentioned that cables placed vertically over each other may be connected in a way to form a suspended arch. This requires but little material for stiffening purposes. No advantage was taken of this circumstance, though, if adopted, it would considerably lighten the weight of the bridge.

Another reduction in the weight of the stiffening girder could be made by omitting the center hinge and making the truss continuous. This would reduce its weight nearly 10 per cent, but as the calculation of such a truss requires the introduction of the elastic line, and could not be checked without considerable labor, I refrained from discussing the same in this communication.

A comparison between a suspension bridge and the proposed cantilever bridge must principally refer to the cost. Assuming the weight of the cantilever bridge to be about 116,000 tons, its cost at 4 cents per pound would be $9,280,000. To this must be added the cost of 1,080-foot approach, because the suspension bridge is so much longer from end to end. If constructed like the western land span, this cost would amount to $440,000.

It appears from the published river profile that the foundation of the west cantilever tower must reach to 200 feet below water level, while that of the suspension bridge tower would probably be less than 100 feet. This would make a difference of at least $1,000,000 in favor of the suspension bridge if the lower portion of the foundation were calculated at the same rate of cost as the upper portion. The end abutments of the cantilever bridge will add $105,000, hence its total cost (exclusive of foundations), as compared with the cost of a 3,200-foot span, will be $9,280,000 + $440,000 + $1,000,000 + $105,000 = $10,825,000, or $2,409,500 less than a suspension bridge of 3,200-foot span, supposing that the assumed weight of the cantilever bridge be approximately correct.

There are some points tending to lower the cost of the suspension bridge or to raise that of the cantilever, for instance, the erection of a suspension bridge after the cables are finished, is much simpler and cheaper than the erection of a cantilever. The latter requires two false works 810 feet long by 150 feet high, and the main span must be erected from the towers toward the middle, while the superstructure of a suspension bridge can, without false works, be erected simultaneously at many places, as the cables form a bridge in themselves to work from at any point.

In regard to the safety factor, if assumed to be 3 in either design, it is relatively of greater value for the suspension cables than for the cantilever truss, because the latter is exposed to impacts while the cable is free from them. A rolling load of 18,000 pounds, on which the calculation of the suspension bridge was based, is an excessive assumption, because the probability of a fully loaded floor decreases proportionately with the length.

For a live load of 18,000 pounds per linear foot of bridge on a 2,100-foot span, one of 15,000 pounds per linear foot would be a full equivalent for a span one-half longer. This item alone would save $226,000 in the cost of cables and anchor chains, not to mention the saving in the stiffening trusses and anchorages.

It should be noticed, also, that the estimated cost of the suspension bridge includes contingencies, while no contingences were considered in the estimate of the cantilever. All these points taken into consideration, it is probable that the actual difference of cost between a cantilever bridge of 2,100-foot span, resting on a pier in the middle of the river, and a suspension bridge of 3,200-foot span, without an intermediate pier, may not exceed $2,000,000.

In the foregoing description I frequently used technical terms and mathematical expressions which are probably of no interest to you, but as I understand that your object is to submit this report to a board of expert engineers, familiar with the science of bridge construction, I know they will find it easier to pass an opinion on what I have endeavored to elucidate when the principles from which the data were deduced are mentioned than if I had merely furnished a table of quantities.

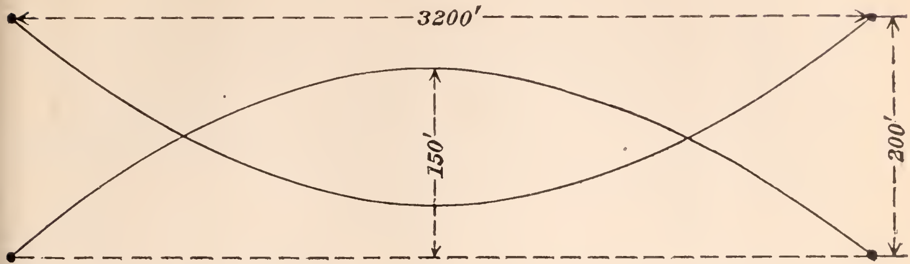
Respectfully submitted,

W. Hildenbrand.

Gustav H. Schwab, Esq.,
Chairman Special Committee Chamber of Commerce.

APPENDIX.—CALCULATION OF WIND BRACING.

In addition to the lateral system between the trusses two storm cables will be stretched under the floor from tower to tower:



These cables should be adjusted to a bearing, without initial strains, at 110°. The contraction of the wire at zero will cause an initial strain of 26,800 pounds per square inch, hence the strain in the cables will be a maximum if the greatest wind would occur at the coldest weather.

If it be the strain per square inch of cable caused by the wind pressure, the total strain in the cable resisting the wind force will be very nearly $26{,}800 + \frac{t}{2}$ and in the other cable $26{,}800 - \frac{t}{2}$, hence if $t = 2 \times 26{,}800$ pounds, the strain in one cable will be 53,600 pounds per square inch, and in the other 0.

Assuming 150 feet to be the deflection of the cables in their normal position, and allowing an extreme side deflection of the floor of 10 feet, the following table gives the conditions of the cables at 0° F.:

Length of cable.	Deflection.	Tension per square inch.	Pressure per linear foot of span, producing the tension per square inch in the preceding column.	Total pressure for entire span.
Feet.	*Feet.*	*Pounds.*	*Pounds.*	*Pounds.*
3215.76	137.7	....	....	....
3218.6	150	26,800	3.14	10,048
3221.4	160	52,600	6.16	19,712
3216.28	140	4,220	0.5	1,600

It follows, from these figures, that a strain of 52,600 pounds per square inch for a side deflection of 10 feet from the normal position, will resist a wind pressure of $6.16 - 0.5 = 5.66$ pounds per linear foot because the 0.5 pound arises from the pressure of the opposite cable, and not from the wind, or a total pressure of 18,112 pounds.

The wind surface was approximated at 42.5 square feet per linear foot, and the wind pressure at 30 pounds per linear foot; hence, the total pressure is 1,275 pounds per linear foot extended over the whole middle span. Assuming a storm cable of 126 square inches section weighing 420 per foot, it will resist a pressure of $126 \times 5.66 = 713$ pounds; hence, the truss chords and lateral system must resist $1{,}275 - 713 = 562$ pounds per linear foot.

The weight of the horizontal-web system was computed at 434 pounds per linear foot. The chord section for the wind truss will, therefore, be 337 square inches.

The deflection (at 20,000 pounds per square inch unit strain) of this wind truss in the plane of the floor beam, and assumed as continuous, if calculated for a load of 562 pounds per linear foot, is greater than 10 feet; hence, the chord section must be increased to reduce the deflection to 10 feet. It will be found that a chord section of 466 square inches per truss complies with this condition, which reduces the unit wind strain from 20,000 to 14,000 pounds per square inch.

On account of the center hinge (hinging the truss vertically) it will be necessary to transmit the wind pressure to the towers by means of the bottom chord alone. The section of one bottom chord of each of the two trusses in the quarter span is

633 square inches, and in the center of the bridge will be about 270 square inches; hence, it will be necessary to add 196 square inches extending over about 300 feet each side from the center.

The wind strain in the quarter span will be found to be 2,585 tons. This is resisted by a bottom-chord section of 633 square inches in each truss, to which should be added 53 square inches extending over 700 feet on each side of the center. With this increase of section, the maximum unit strain in the bottom chord would be 26,000 pounds, under the supposition of the improbable case that the greatest wind strain would coincide with the maximum vertical distortion of the truss under a one-sided load. The additional weight, corresponding to the mentioned increase of section in the bottom chord, amounts to 418 pounds per linear foot of bridge.

The floor beams, which are supposed to be 30 feet apart, must resist the pressure of the cables, amounting to $126 \times 6.16 = 776$ pounds per linear foot, or of 23,280 pounds per floor beam. This will require an increase of 1.16 square inches, or 370 pounds of metal per floor beam, equal to 13 pounds per linear foot.

The total weight of the wind system, therefore, is:

	Pounds per linear foot.
Lateral-web system	434
Additional of bottom chords of trusses	418
Increase in floor beams	13
Storm cables	840
Total	1,705

In the original estimate of weight the total weight of wind bracing was calculated to be 1,240 pounds per linear foot, based on the supposition of a continuous truss which could transmit the wind pressure to the towers without requiring additional chord sections.

The suspender weight was calculated for a unit strain of 20,000 pounds, while 40,000 pounds would be a moderate strain if the suspenders be constructed of wire rope, reducing the assumed weight one-half.

The weight of wind bracing and suspenders should therefore be corrected as follows:

$1{,}705 + 555 = 2{,}260$ lbs., costing 865 lbs. $\times$ 3,100	1,340 tons, at 4 cents =	$107,260
$840 + 555 = 1{,}395$ lbs. $\times$ 3,200	2,232 tons, at 8 cents =	357,120
	3,572	464,380

This is 188 tons and $137,000 less than given in the estimate on page 67.

The tower columns were calculated for a unit strain of 12 tons on top and 10 tons at the bottom; hence, no addition was made for wind strains, which are insignificant compared with the direct strains from the weight of the bridge.

W. HILDENBRAND.

APPENDIX C².

STATEMENT OF MR. W. HILDENBRAND TO THE BOARD OF ENGINEERS.

NEW YORK, *July 20, 1894.*

GENTLEMEN: I beg to submit to you herewith a modified plan and estimate for a suspension bridge across the Hudson River at Sixtieth street. It differs from my former design, offered to the inspection of your honorable Board on July 17 by the chairman of the special committee of the Chamber of Commerce, in being adapted to the correct profile of the river and complying with your instructions as to unit strains in different parts of the superstructure and the admissible pressure on the foundation:

The following are the principal data of this plan:

Total length of bridge	feet	4,310
Span from center to center of towers	do	3,310
Width of towers at base	do	180
Clear span between piers (=clear waterway between pier-head lines)	do	3,130
Width of tower at high water, center to center of columns	do	131.5
Length of tower at high water, center to center of columns	do	340
Width of tower at floor line	do	100
Width of tower at top	do	70
Clear span of cable in middle span	do	3,240
Deflection of cable at 55° F	do	400
Deflection of cable at 0° F	do	397.63
Deflection of cable at 110° F	do	402.30
Lengths of cable in center span at 55°	do	3,369.1

Lengths of west back cable	feet	1,611.7
Lengths of east back cable	do	903.2
Total lengths of cable, anchorage to anchorage	do	5,884
Rise and fall of cable for a difference of temperature of from 0°–110°	do	4.67
Deflection of west back cable from a straight line connecting top of tower and anchor pin:		
For dead load	feet	32.1
For dead and live load	do	21.6
Deflection of east back cable from a straight line connecting top of tower and face of anchorage:		
For dead load	feet	8
For dead and live load	do	5.3

Depression of center span when fully loaded arising—

	Feet.		
From elongation of center cable	1.77	do	6.3
From elongation of west back cable	0.85		
From elongation of east back cable	0.47		
From change of deflection in west back cable, causing the saddle to move forward	0.92		
The same of the east back cable	0.06		

(This depression could be reduced one-half if the cables were connected with the land piers by suspenders and held down.)

Total rise and fall in center of bridge under extremes of temperatures and loads	feet	10.97
Camber of floor at 55° F	do	9
Grade of floor at 55° F	per cent	1.11
Grade of floor at 0° F. (camber 11.37', less contraction of tower = 0.19)	do	1.38
Grade of floor at 110° F. (camber = 6.7 + elongation of tower = 0.19)	do	0.85
Camber of floor at 110° if fully loaded (0.4 + elongation of tower)	feet	0.59

These figures show that the floor will never sink below level and that the maximum grade is less than 1½ per cent. It should also be noticed that this grade extends only over a few feet near the towers, and diminishes rapidly when a train proceeds toward the center of the bridge.

The height of the towers is:

	Feet.
From high water to under side of bridge	150
Thickness of floor	8
Camber of floor	9
Bottom of cable above floor	2
Deflection of cable at 55°	400
Extra height for the support of two tiers of cables	40
Total	589

The live load was assumed to be 18,000 pounds per linear foot. The unit strain in stiffening trusses does not exceed 15,000 per square inch for reversed stresses. The unit strain in the cables was restricted to 50,000 pounds per square inch for wire having an ultimate strength of 180,000 pounds. The stiffening truss was calculated to resist a live load of 18,000 pounds per linear foot, covering one-half of the span, while the opposite half be unloaded.

It will require a moving load of 2,400 pounds per linear foot to deflect the dead mass of the bridge (without regard to any stiffening) 3.8 feet in one-quarter span and raise it 3.7 feet in the opposite quarter.

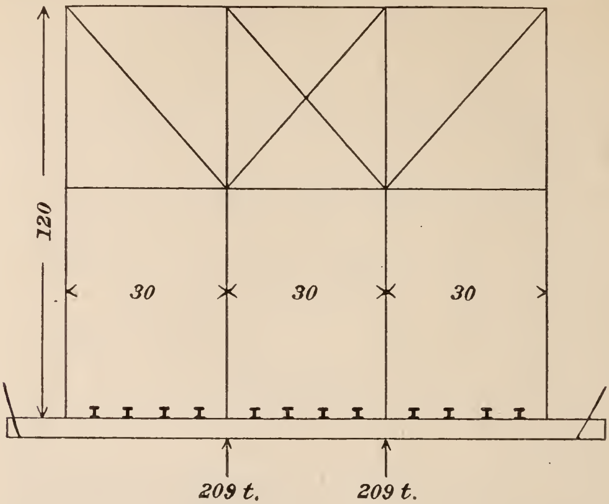
This distortion of the floor causes a grade of less than 1 per cent. Hence this stiffening girder was calculated for a moving load of 15,600 pounds.

To comply with this condition and not to strain the metal above 15,000 pounds per square inch, the height of the truss was chosen to be 120 feet giving the following dimensions:

Maximum chord, section of either top or bottom chords	square inches	1,395
Average weight of both chords	pounds per linear foot	8,060
Average weight of web system	do	4,220
Total	do	12,280

It will be found that the deflection of this truss for a load of 15,600 pounds per linear foot is 2.35 feet if the elongations and contractions of the web members be neglected; hence its true deflection will about coincide with the distortion of the cable under a load of 2,400 pounds per linear foot.

The intermediate floor-beam suspenders and cross-bracings were calculated for a local load of 200 tons, in addition to the dead weight on each suspender.



Allowing a unit strain of 20,000 pounds per square inch, the weight of these parts will be 1,332 pounds per linear foot.

The question was investigated whether there was an appreciable inclination of the floor beams if three tracks on one side of the axis of the bridge were fully loaded in one-half of the spans, and if the opposite three tracks were loaded in the other half of the span. For instance, a load of 9,000 pounds per linear foot on the three right-hand tracks will depress the right end of the floor beams 1.37 feet and the left end 0.53 foot. The same load on the left three tracks in the opposite half span will raise the right ends of the former floor beams 0.53 foot, and will depress the left ends 1.35 feet; hence, the greatest inclination of a floor beam is 1.68 feet in 100 feet, which is hardly noticeable.

The two suspenders of one floor beam were calculated with a unit strain of 50,000 pounds per square inch for the dead load, plus 600 tons, assuming six 100-ton locomotives to meet on one floor beam.

The estimated weight is 776 pounds per linear foot. The aggregate load to be sustained by the cable is:

	Pounds per linear foot.
Moving load	18,000
Stiffening trusses	12,280
Platform	5,400
Projecting ends of floor beams	300
Intermediate floor-beam suspenders and sway braces	1,340
Lateral wind bracing	860
Storm cables	840
Suspenders	780
Cables	13,000
	52,800

Total load on cables, $39,800 \times 3,210 + 13,000 \times 3,240 = 84,939$ tons.

Tension in cable at one-eighth deflection, 94,960 tons.

Allowing 25 tons strain per square inch, it requires 3,800 square inches, or 81,270 No. 3 wires (0.244 inch diameter).

Dividing the number of wires into 16 cables, one cable will contain 5,080 wires and will have a diameter of 20¾ inches.

The tension in the east anchor chain will be 94,700 tons. The tension in the west anchor chain will be 88,500 tons; average tension, 91,600 tons; requiring 9,160 square inches and weighing 39,700 pounds per linear foot.

	Tons.
Total weight of anchor chain for a length of 510 feet	10,120
Weight of anchor plates	1,580
	11,700

The towers were calculated for the combined maximum load and wind strains at 12 tons per square inch.

	Tons.
Weight on top of tower (requiring 7,080 square inches)	84, 940
Weight of tower	15, 110
Weight of land truss resting on tower	1, 400
Wind strains	1, 350

Pressure at base of tower, 102,800 tons, requiring 8,566 square inches. Average section, 7,823 square inches.

Weight per linear foot, 52,150 pounds.

Total weight of two towers, 579 feet high, 30,200 tons.

The pressure on the rock foundation of the east tower will be 247,380 tons, and the buoyancy of the pier cylinder 90,250; hence the foundation must contain 15,713 square feet in order to resist a pressure of 157,130 tons.

This area can be procured by sinking eight cylinders of 50 feet diameter, filled with concrete, one for each tower column. The total mass of foundation work will amount to 2,081,300 cubic feet. The pressure on the west tower is less, owing to the light inclination of the back cable, hence concrete-filled cylinders of 47 feet diameter will answer the conditions of the foundation.

The total pressure is 137,710 tons, requiring a foundation mass of 1,958,600 cubic feet.

The east land pier, supporting two independent 400-foot truss bridges, requires a foundation mass of 100,400 cubic feet, requiring four cylinders of 20 feet diameter. As regards the pressure on the foundation, the cylinders might be smaller, but as they must be sunk to a depth of about 80 feet, a smaller diameter seems not to be advisable. No land spans were designed for the west side, as the west tower coincides with the position of the west end abutment of the cantilever bridge; hence, whatever construction be used for this approach would be common for both designs.

The west anchorage is entirely in rock 200 feet below the surface, requiring 13,200 cubic yards of rock excavation, 9,000 cubic yards of concrete filling, and 25,000 cubic yards of masonry.

Estimating the rock excavation at $4, the filling at $6, and the masonry at $14 per cubic yard, the cost of this anchorage will be $456,000.

The east anchorage is also partially in rock, which according to the contour of the rock strata will probably be found at a depth of 30 to 35 feet below high water. It requires, therefore, 6,800 cubic yards of rock excavation and filling, 80,000 cubic yards of earth excavation, and 80,000 cubic yards of masonry.

Estimating rock excavation at $3, earth excavation at 50 cents, concrete filling at $6, and masonry at $14, the cost of this anchorage will be $1,221,000.

The following is the calculated weight and estimated cost of the bridge:

Stiffening truss, 12,280 × 3,210 = 19,710 tons at 4 cents	$1, 576, 700
Platform, 4,200 × 3,310 = 6,950 tons, at 3¼ cents	451, 800
Intermediate floor-beam suspenders and sway bracing, 1,340 × 3,210 = 2,150 tons, at 4 cents	172, 000
Towers, 30,200 tons, at 4¼ cents	2, 576, 000
Woodwork and track, 4,310 feet, at $24	103, 400
Anchor chains and plates, 11,700 tons, at 3½ cents	819, 000
Cables, 13,000 × 5,884 = 38,250 tons, at 7 cents	5, 354, 400
Wind laterals and additional weight in bottom chord and floor beams, 860 × 3,210 = 1,380 tons, at 4 cents	110, 400
Storm cables, 840 × 3,210 = 1,350 tons, at 8 cents	235, 700
Suspenders, 780 × 3,210 = 1,250 tons, at 8 cents	200, 300
East land span, 13,950 × 800 = 5,580 tons, at 4 cents	446, 400
East land pier, 150 feet high = 420 tons, at 4 cents	33, 600
Anchorages	1, 760, 000
Total superstructure, 118,940 tons	13, 830, 700
Total foundation mass = 4,140,000 cubic feet, estimated at 50 cents, will amount to	2, 070, 000
Total bridge, 4,310 feet long	15, 900, 700

The proposed cantilever bridge is but 4,120 feet long, and if its weight, as assumed in my former report be correct, and the foundation be calculated on the same basis, its cost would be about $12,665,000, or $3,235,000 less than a single span suspension bridge.

If the cantilever bridge was calculated, as I understand it was, for a unit strain of 20,000 pounds per square inch of metal, I beg to draw the attention of your honorable Board to the fact that this circumstance puts the comparison between the two

bridges on two different bases. According to Cooper's specifications, soft steel would not reach a strength of over 60,000 per square inch, and medium steel would have an average strength of 64,000 pounds, hence a strain of 20,000 per square inch is equal to a factor of not over 3.2. The steel wire of which the cables of the suspension bridge are to be made will have a minimum strength of 180,000 pounds, hence a unit strain of 50,000 pounds is equal to a safety factor of 3.6. If this factor be placed at only 3.2, as in the cantilever bridge, the admissible unit strain would be 56,000 pounds and the saving in the cost of cables $600,000, and in towers and anchorages $180,000.

In regard to employing a unit strain of 20,000 pounds for the stiffening truss, as assumed in my first report, it seems at first sight high on account of working in compression as well as in tension, which induced your honorable Board to restrict this strain to 15,000 pounds per square inch. Personally I believe that a strain of 20,000 will give ample safety, because the reverse strains will occur only at long intervals. But even with a strain of 15,000 the weight of the stiffening construction could easily be reduced 1,800 to 2,000 pounds per linear foot by connecting each pair of cables in a vertical plane in a way to form a suspended arch. The saving of weight in the superstructure, cables, towers, anchorages, and foundation work would, in that case, amount to at least $1,600,000, reducing the difference of cost between the two designs, cantilever and suspension bridge, to about $1,500,000.

Respectfully submittted.

W. HILDENBRAND.

To the BOARD OF ENGINEERS,
New York and New Jersey Bridge.

APPENDIX D.

NORTH RIVER BRIDGE TO HOBOKEN.

[Designed by G. Lindenthal, Chief Engineer.]

GENERAL.

Location.—It is shown on the attached map, Exhibit A. Under the charter the company could locate the bridge anywhere over the Hudson River between the Battery and the northern New York City limit. The location at Hoboken was advised by the principal railroad interests as the most direct entrance into New York. It has also the advantage of accommodating local travel to Hoboken and Jersey City Heights, the revenue from which will be a large item.

From a purely engineering point of view a location farther up, where the rocky bluffs are close to the water's edge, would have been preferable. It would have cheapened the construction of the bridge (with a single span under the charter) by several million dollars. The managers of the company, however, considered the business advantages of locating at Hoboken, and the larger revenue therefrom as far outweighing the cheaper cost of construction on a location farther north.

The location was approved by the Secretary of War December 29, 1891.

The New York anchorage for this location had to be placed at the intersection of Twenty-third street and Tenth avenue, where the borings indicated the rock at 22 feet below the surface. It is the old natural river shore. From this line towards the river is made ground, and the rock dips rapidly, reaching a depth of 190 feet at the foot of Twenty-second street, where the New York tower is located.

Rocky bottom (whether solid or bowlders is not definitely known yet), overlaid with sandy clay, containing smaller bowlders to a depth of 26 feet, is found to exist for the New Jersey anchorage. The New Jersey tower will require less than one-half the depth of the New York tower for a foundation.

The river between pierhead lines is here 2,740 feet wide. The New Jersey tower is located close to the New Jersey pierhead line. The New York tower is located 150 feet back of the New York pierhead line, to shorten the New York end span, and to avoid deeper foundations. The resulting span is 3,100 feet center to center of towers.

The total length of the bridge between anchorages is 6,800, and including the anchorages, 7,340 feet.

Capacity of bridge.—It is based on the view that the bridge must derive its largest business from suburban traffic at low rates. The passenger travel to distant points, together with the freight traffic likely to go over the bridge, would not pay on the investment.

The bridge, as a lateral outlet for the masses, must have a large track capacity, on which the company has been at great pains to get the advice and estimates of railroad managers familiar with the situation.

It is not necessary to give here the estimate of the number of trains capable of handling over the bridge, but only to explain that the in and out going tracks will be connected by loops at the New York terminus to avoid switching and delay, and to save the costly space for a switching station. As far as the New York passenger terminus is concerned, it will be as for a 4-track railroad doubled on itself.

A switching and cleaning yard will be provided in the New Jersey meadows.

The number of tracks over the bridge will be 8, namely, 2 tracks for suburban travel, 2 tracks for through passenger and express trains, 2 tracks for freight, and 2 tracks for electrical railroads.

During certain hours in the morning and evening the freight tracks will be available also for suburban business.

The suburban loaded trains coming in the morning will turn around the loop and go back empty to their respective yards. The reverse process will take place in the evening hours.

Since the largest growth of business will come from the suburban traffic, the structure is so designed as to permit of an addition of 6 rapid-transit tracks on a second deck.

The anchorages and towers will be constructed for a capacity of 14 tracks, as shown below. All other parts of the structure will be erected for a capacity of 8 tracks.

The additional cost of the heavier anchorages and towers is such a small percentage (about 9 per cent) of the total cost of the structure that it cuts no figure in the large cost of the entire undertaking.

Following is a brief description of the salient features, and of the dimensions and quantities of material in the structure:

(1) *Anchorages.*—Pull on each anchorage.

From dead load, 8 tracks (15 tons per linear foot of superstructure), 58,000 tons. From the assumed live load (12 tons per linear foot of bridge on 8 tracks), 46,000 tons. Total, 104,000 tons.

Provision is made for a future increase to 14 railroad tracks, for which the pull from dead load (19 tons per linear foot of suspended superstructure) would be 73,000 tons, and for live load (18 tons per linear foot of bridge), extreme limit, 65,000 tons. Total, 138,000 tons, maximum.

Total net section of anchorage steel bars (60,000 ultimate average strength) in each anchorage, 10,000 square inches.

Anchor platforms of steel, 90 feet below the pavement.

Weight of masonry and rock, and filling of stone, gravel, and sand, of each anchorage	tons	480,000
Assumed coefficient of friction for masonry on foundation		0.60
Minimum resistance of anchorages against sliding	tons	288,000
Total bearing area of anchor platform and anchor chains against masonry in each anchorage	square feet	17,000

(For quantities, see estimate below.)

Quantities in each anchorage (to grade line).

Excavation	cubic yards	30,000
Concrete masonry	do	96,000
Ashlar granite facing	cubic feet	440,000
Filling for weight, rock, gravel, sand, and tamped earth	cubic yards	160,000
Steel in anchors and anchor bars	tons	6,200
Asphaltum for metal bearings	cubic feet	1,000

NOTE.—The structure above the grade line of the anchorage is a building with an open court or yard over the tracks—this building to contain way station (for electrical cars), accessible by elevators from the street, the upper part to contain offices (for renting purposes) with their windows out upon the inclosed space or court.

The outside of this building corresponds in architecture with the base of the anchorage, but the weight of the building is not considered (in the above given weight) as a part of the anchorage.

(2) *Towers.*—The tower bases are hollow, of masonry, reaching 40 feet below high water and extending 30 feet above high water.

The construction of the tower foundation on New Jersey side, 90 feet down to rock, is by the usual pneumatic method, using a wooden caisson, 175 by 335 feet, with two hollow spaces, each 90 feet square, where there is no pressure from the steel columns, the air chamber, after reaching firm bearing, to be filled with packed sand and gravel, and with concrete where necessary.

The wooden caisson is of cellular construction; one-third of the section consists of gravel and sand filling.

Bearing area, 40,000 square feet.

Pressure on foundation 80,000 tons from tower base, after deducting displacement; 76,500 tons superstructure and steel tower, complete for 14 tracks; 61,000 tons extreme live load from 14 tracks; total, 217,500 tons.

	Tons.
Pressure, per square foot	5.425
From wind pressure of 3,000 tons on tower, on lee side	.200
Maximum (per square foot)	5.625
From dead load alone (per square foot)	3.92

Maximum pressure on timber, 80 pounds per square inch.

The New York tower foundation, 190 feet down to rock, is of a different construction. An open, braced caisson, or cofferdam, with lower edge conforming to contour of rock, as obtained by borings all around, 350 by 180 inside, of wood and iron, 10 feet thick, filled with gravel, is first sunk and the inside dredged out down to rock, which is leveled off with concrete in bags, and finely broken stone, below water A hollow-spaced wooden crib, 345 feet by 175 feet and 150 feet deep, is built up floating inside the caisson. Two large hollow spaces, 75 feet square, enlarging towards the top to 90 feet square, are spared out in the center of each half tower, where there is no pressure from the steel columns. Masonry below water is also built with hollow spaces. The whole mass of foundation is calculated to float during construction, so that all masonry can be done above water till the whole settles down evenly upon the leveled foundation. All hollow spaces in the wooden crib are then filled with gravel and sand, and in the masonry, with concrete.

Maximum pressure on rock foundations: New York tower, 130,000 tons, tower base, after deducting displacement; 75,000 tons, superstructure and steel tower; 61,000 tons, extreme live load. Total pressure, 267,500 tons on 50,000 square feet, or 5.35 tons per square foot; from extreme wind pressure, 0.26; maximum pressure, 5.61 tons per square foot; from dead load alone, 4.13 tons per square foot; maximum pressure on timber, 80 pounds per square inch.

Each steel tower has 16 columns, with a total cross section of hard steel (100,000 pounds ultimate per square inch). At the top, 16×515 square inches $=$ 8,240 square inches. At the base, 16×580 square inches $=$ 9,280 square inches.

Diameter of columns, 8 feet at top; 9 feet at bottom.

(For quantities, see estimate below.)

	Pounds.
Compression per square inch of steel from dead load of only 8 tracks, and from maximum bending moment	12,500
From maximum live load of only 8 tracks	9,600
Total	22,100
From dead load in the future of 14 tracks and from maximum bending moment	15,300
From maximum live load of 14 tracks	14,400
Possible maximum total	29,700

The elastic limit of the 100,000 pound steel is 60,000 pounds. Buckling strength of steel columns is 54,400 pounds per square inch.

With 16 trains of ordinary size (600 tons each) on the bridge, the total compression per square inch in the tower columns will not exceed 13,500 pounds per square inch for 8 tracks superstructure.

For 14 tracks superstructure the total compression from dead and live load will rarely reach 17,000 pounds per square inch.

The above considered maximum compression of 29,700 pounds per square inch may never occur in the life of the bridge. It would require a live load of 122,000 tons, equal to about 3,000 loaded freight cars.

The temperature strains in the cable do not affect the towers, but the towers themselves are exposed to bending strains from temperature differences in the columns, exposed to the heat of the sun, and others being in the shade. The computed deflection of tower tops from this cause, for differences of 30° F., is 2 inches.

The deflection of tower tops lengthwise with bridge from difference of load effects will not exceed 8.1 inches.

Quantities in New Jersey tower base (90 feet below high water).

Excavation	cubic yards..	105,000
Timber	M..	20,000
Iron	tons..	460
Concrete below water	cubic yards..	3,000
Gravel and sand filling	do......	40,000
Masonry of hard-burned brick.		
Concrete and granite facing and coping	cubic yards..	56,000

Quantities in New York tower base and cofferdam (192 feet below high water).

Excavation	cubic yards..	360,000
Timber	M..	65,000
Iron	tons..	2,340
Concrete under water	cubic yards..	1,000
Broken stone	do......	3,000
Gravel filling	do......	180,000
Masonry, the same as for other tower	do......	56,000
Weight of steel for each tower:		
Sixteen columns, with base plates and cable bearings	tons..	7,220
Lattice, horizontal, wind, and longitudinal bracing	do...	4,060
Cable chambers on top and upper bracing between towers	do...	940
One tower	do...	12,220
Two towers	do...	24,440
Quantities for each pedestal:		
Excavation	cubic yards..	2,000
Masonry, concrete, with granite facing and coping	do......	1,200
Steel columns, with bracing and vertical anchorage	tons..	170

(3) *Cables.*—The 4 cables are computed with a factor of safety of 3 for a dead load of 15 tons per linear foot, and for a moving load of 12 tons per linear foot, covering the entire bridge. Maximum total load, 27 tons for 8 tracks.

In ordinary operation the moving load will rarely reach 3,000 tons for the middle span on all 8 tracks.

The 4 cables are composed of pin-connected wire links. Each wire link is made up to accurate length of parallel wires looped around flanged steel shoes, bored out to fit the pins. The pins are 16 inches diameter and hollow. The steel wire is No. 3, Birmingham gauge (0.259 diameter), and has an ultimate strength of 180,000 pounds per square inch (9,600 pounds per wire).

Wire links have the advantage of accurate work and close inspection in the shop, quick erection, and testing to destruction, so their accurate value may be known. Wire links permit also of variations in the cable section, as needed.

The length of the wire links varies from 50 feet at the center to 54 feet at the towers.

The horizontal panel length is 50 feet throughout.

The links contain from 400 to 800 wires each, according to position in the cable, and on the pins.

The vertical distance of the cables is 55 feet from center to center throughout, i. e., the upper and lower cable have the same curvature.

Each cable is composed of 3 chains, one above the other, coupled vertically at the pins. The vertical couplings consist of one-half inch steel plates between links, sufficiently strong transversely to transmit the increment of the web strains into the chains of the cable.

The middle chain has alternately 9 and 10 links, and the upper and lower chains have each alternately 7 and 8 links.

Total number of wires in each cable: At the tower, 18,400; solid metal section, 975 square inches; ultimate strength, 87,700 tons. At the dip, 16,900; solid metal section, 890 square inches; ultimate strength, 80,000 tons.

The dip of the cables is one-tenth of the span, 310 feet.

The cables are so arranged that for an addition of 6 tracks (to a total of 14) the wires in each cable can be increased in the future up to 25,000 by the addition of wire links.

The chains of wire links are surrounded by a water-tight removable shell (or envelope) of corrugated steel (one-eighth inch thick) 9 feet in diameter, as a protection against the weather.

The bracing between the cables is of rolled steel.

The verticals consist of two latticed 20-inch eyebars, varying in cross section for one member from 140 to 80 square inches.

The diagonals consist of sets of adjustable rods, from 2 to 3 inches square, with screw ends fitting into eyebar saddles, having their bearings on the cable pins of the middle chain. The aggregate section of the square rods varies for one member from 206 to 92 square inches.

The average weight per linear foot of each suspended rib, including wire links, steel shoes, pins, and steel envelope, is 7,200 pounds; bracing between cables, 1,300 pounds; total net weight of one rib average per linear foot, 8,500 pounds.

Compared with one of the four arch ribs of St. Louis bridge: Steel tube flanges, 635 pounds average per linear foot; web, 285 pounds average per linear foot; total, 920 pounds.

Or, compared with one of the six arch ribs of the Harlem bridge: Flanges, 850 pounds average; web, 190 pounds average; total, 1,040 pounds.

In the St. Louis bridge the web is 30 per cent of the total weight of the rib; in the Washington bridge, 19 per cent, and in the North River bridge only 15 per cent of the total weight of the rib.

The two ribs can resist a bending moment of 1,600,000 foot tons before the permanent tension in the upper cables from the dead load would be nullified. (Wire-link cables can not take compression, and overstraining from bending is next to impossible).

Calculation shows that to produce this bending moment trains, would be required side by side, 1,200 feet long, aggregating 10,800 tons moving over the bridge.

It would be next to impossible to bring such a load and such an extraordinary assemblage of double-headed trains upon the bridge.

The assistance of the stiffening trusses is here entirely disregarded, also that part of the live load directly absorbed by the cables by reason of their deflection.

The calculated maximum deflection, at one quarter the span, from this extreme load is 2.4 feet. In every-day operation the load effects and deflection will probably not reach 10 per cent of this extreme.

The suspended ribs, with a depth of one fifty-sixth of the span, are therefore stiff and strong enough to resist deformation, without stiffening trusses, from the heaviest moving load, and being in stable equilibrium are more rigid than the compression ribs with fixed ends of the St. Louis arch bridge with a depth of one forty-fourth of the span, which are in unstable equilibrium and subject to compression and tension alternately.

The suspended ribs are assumed to be hinged at their ends in the anchorages and on the towers. This assumption is severely carried out in the construction. The exact equivalent for the hinge on the towers is a toggle joint, made of short wire links.

The diagonals of the cable bracing during erection remain loose. After erection of complete superstructure, and with the cables therefor in perfect equilibrium from dead load, equally divided upon the four cables, the diagonals are adjusted and receive a slight initial tension.

The cable bearings on the towers are fixed, i. e., can not slide, but they can accommodate themselves to the hinge movement of the toggle joint. The bending strains in the towers (from the difference of temperature, and from load effects in the cables) are allowed for.

The temperature strains in the cables are relatively small. The steel envelopes protect the wires against the direct rays of the sun and against uneven heating.

The bending moments from live load in the long end spans (having a dip of $\frac{1}{8.4}$) would be greater than in the middle span.

To reduce them to the same limit, the end spans are in the middle provided with anchor columns, reaching to masonry pedestals below. These columns bear no part of the dead load. They are counterweighted in the pedestals in such manner as to be affected only by an excessive concentrated live load, in which case a positive or negative reaction (of $\pm$ 6,000 tons, according to position of live load) is produced and accurately known. The computation of the bending strains is thereby much facilitated.

(NOTE.—This is the present arrangement, but a change to a fixed hinge, as more economical still, is under consideration).

The weight per linear foot of the suspended structure in the end spans is brought within the same limits as for the middle span.

The position of the anchorages and lengths of end spans are dictated by local conditions. If the anchorages could have been placed nearer to the towers it would have reduced cost of bridge very much, as shown below.

The arch ribs are cradled 6.5 per cent, i. e., inclined toward each other. They are 160 feet apart on top of towers and 120 feet at the middle of span.

No wind bracing is required between the cradled cables.

The suspenders are of bundled steel wire ropes, 35 square inches solid section. Maximum tension, with 14 tracks, on one suspender, 700 tons.

Assuming E = 29,000,000, the middle span would deflect under the assumed maximum load of 12 tons per linear foot, which may *never* occur.

From elongation of cables in middle span	inches	48
From bending of towers	do	21
From + 65° F., as a maximum, affecting only the middle span	do	21.7
Maximum deflection	do	90.7

Normal camber 15 feet = 180.0 inches for middle temperature at + 50° F.

	Pounds per square inch.
Tension in the cable wires from dead load alone	32,500
From temperature bending moment	6,200
From full live load (12 tons per linear foot)	26,000
Maximum at center of span	64,700
Maximum from bending moment at the quarter	65,000
Lowest elastic limit of 180,000 pounds steel wire assumed at	85,000
But more likely to run up to	120,000

The maximum deflection, at the quarter from one-sided loading, of 29 inches is equal to a change of six-tenths per cent in the grade on the middle span.

Track platform and wind girders.—The panels are 50 feet long. Stringers placed directly under the rails, which rest on wooden block cushions between steel guard rails, open on the sides to let cinder and snow fall through; no wooden ties at all are used.

The space between steel guard rails is stretched over with a taut wire netting, weighing 2½ pounds per square foot.

Stringers and floor beams, 5 feet deep, dimensioned for locomotives weighing 150 tons with tender.

The lower floor beam carries 8 tracks, and has 3 intermediate supports from a cross arch above, for which it acts as the tension member between arch footings.

The cross arch is 65 feet high and placed at every panel point. It acts also as cross bracing between the lower and upper wind trusses.

The cross arch is further to carry an upper floor beam for a second deck of 6 rapid-transit tracks to be added when needed in the future.

On top of the cross arch is the promenade, 20 feet wide with wooden flooring.

The lower wind truss is in the plane of the lower floor for 8 tracks. The upper wind truss is placed below the promenade. Both are 115 wide, center to center of chords, equal to one-twenty-sixth of the distance between the towers.

Both wind trusses have the same chord sections (180 square inches each for 100,000 pounds steel) from end to end (anchorage to anchorage).

The wind trusses are proportioned as horizontal continuous girders of uniform chord section, with a consequent great saving of metal for the chords. Their ends in the towers can slide and adjust themselves to temperature changes, but are arranged to resist vertical and horizontal bending moments at the towers.

The ends of the wind trusses at the anchorages are firmly anchored into the masonry, to also resist end-bending moments.

The assumed wind pressure is 2,400 pounds per linear foot of superstructure.

Maximum stress in chord ± 36,000 pounds per square inch and maximum stress in diagonals + 30,000 pounds per square inch from maximum wind pressure.

The chord sections of the horizontal wind trusses are utilized also in two vertical stiffening trusses, in connection with the floor system and cross arches, as an aid in distributing the concentrated train loads upon the braced cables, although not necessary, as stated above.

The cable ribs are so rigid that the aid of the vertical trusses amounts to only 10 per cent; i. e., 90 per cent of the deforming effect from moving load goes into the arch ribs and 10 per cent into the stiffening trusses for equal values of deflection.

Erection.—The construction of the anchorages and the erection of the steel towers offer no new features, except large size.

All steel work, except wire loops, can be done with existing plant. The plant for making wire loops will be simple and not expensive. There will be 9,300 wire links, and it is estimated that on one machine two links can be furnished per day on an average, which would require sixteen months on ten machines for the 9,300 links required.

As for manufacturing capacity of wire, there are several large works equipped for it, and one of them offers to alone furnish all the wire (over 40,000 tons) in one year without extra effort.

The links will average 4 tons each, and the cost of making the links is estimated at five-eighths cent per pound, including oiling. (The wires will not be galvanized).

The erection of the superstructure will be similar to that for the Brooklyn bridge,

i. e., without false works. First, a temporary footbridge of wire ropes over the towers, strong enough to carry one set of wire links of permanent cables.

Then, erection of first wire links, suspended temporarily from the footbridge above, alternating one and two links, commencing at each tower, toward center and anchorage, respectively, until first set of wire links is connected up from anchorage to anchorage for each side. Thereon all following wire links are erected and pushed upon the pins without further support from the footbridge. The wire ropes of temporary footbridge are used afterwards for the suspenders.

The wire links, being all made in the shop to exact length, the same as eyebars, the erection of the cables can proceed without regard to the weather and without delay from adjustment; 160 erecting gangs, if needed, can be employed at one time simultaneously in the erection of the four cables, which can be completed in less than ten months.

The erection of the other parts of the superstructure is the same as for Brooklyn bridge.

It is immaterial what form the cables assume during erection. The adjustment of the diagonal bracing between the cables and of stiffening trusses takes place only after the entire superstructure down to the rails is in place.

The time of construction is estimated at four years, namely, two years for anchorages and tower bases, nine months for steel towers, and fifteen months for cables and superstructure.

Weights of superstructure, 6,800 feet long, between anchorages:

(1) 8 tracks:			
100-pound rails on wooden blocks.			
Steel guard rails and wire netting.			Tons.
200 pounds per linear foot of track			5,440
(2) Floor construction, per panel of 50 feet:	Tons.		
8 pairs of stringers	88		
1 cross arch, with suspended floor beam, including verticals for stiffening trusses	73		
2 sets of hanger eyebars for cable suspenders	7		
4 wind chords	65		
Horizontal wind bracing (average)	12		
Stringers and hand rail for promenade	7		
Diagonals of stiffening trusses (average)	8		
Total per panel	260	Tons.	
Per linear foot of bridge, 5.2 tons		35,360	
(3) Suspenders of wire ropes		1,900	
(4) Two cable ribs:			
Cables with pins, shoes, couplings, and shell	48,960		
Bracing between cables	8,840		
		57,800	
Weight of superstructure, metal			95,060
Weight of superstructure, metal and track			100,500
Average dead load of superstructure, per linear foot of—			
Metal			13.95
Track			.80
Flooring of promenade / Telegraph wires			.25
			15.00

$\frac{15.00 \text{ tons.}}{8} = 1.875$ tons dead load per linear foot of track, which is the same as for a 400-foot Whipple truss span.

Total amount of steel in—

	Tons
Anchorages	12,400
Towers	24,780
Superstructure	95,060
	132,240

Weight of steel per linear foot of entire bridge, 7,340 feet long: $\frac{132{,}240}{7{,}340} = 18.02$ tons, or 2.25 tons per linear foot of track.

The addition of six rapid-transit tracks (for a live load of 1 ton per linear foot of track), whenever needed in the future, will require 17,500 tons wire links and 10,200 tons of other steel work, 27,700 tons, total addition, at a cost of $2,500,000.

If the bridge had been located at some point north, where the rocky bluffs come close to the river, the length would have been reduced to about 5,500 feet, a saving in length of over 1,800 feet.

For suspension spans of 3,000 feet, a difference of 200 feet more or less span does not greatly affect the cost per running foot.

The cost of bridge would have been reduced:

(1) In the anchorages, on account of the natural rock bluffs, at least....	$1,000,000
(2) In the towers, probably nothing.	
(3) In the superstructure, 1,800 feet shorter, about........................	2,200,000
	3,200,000

Hence the bridge for eight tracks would then have cost only about $17,800,000.

There is, however, an offset of 1,800 feet approaches against such saving, which, at $400 a linear foot, would cost $720,000, thus making the saving only about $2,480,000.

Six per cent interest thereon would have amounted to $148,800 yearly.

The location at Hoboken holds out the certainty of greater revenue, from local traffic alone, by ten times the amount of the interest on the saving in cost of a shorter and cheaper location further north.

The cost of the land, of the approaches, and of the terminal station are greater than the cost of the bridge. There is, further, the interest account during construction and the legal and administration expenses.

July 18, 1894. G. L.

APPENDIX E.

THEORY OF THE CONTINUOUS STIFFENING TRUSS WITH ENDS ANCHORED DOWN BUT NOT FIXED IN A VERTICAL PLANE.

The ends of this stiffening truss are free to change their direction, but not their elevation, in a vertical plane, and the truss is continuous from end to end. The theory of this truss has been published for a considerable number of years in standard engineering works, and its principal formulæ only will be given without detailed demonstration:

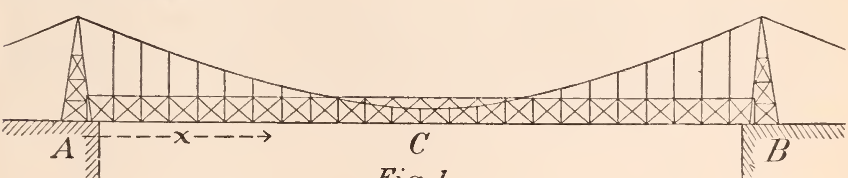


Fig. 1.

The moving load will be taken as passing on the span from the left end A Reference will be made to Fig. 1 in the analysis and the following notation will be employed:

Length of stiffening truss between centers of end supports..................	l (feet)
Moving load per linear foot	w
Uniform upward pull (supposed to be distributed over l) per linear foot....	p
Reaction of truss at A (upward under conditions taken).....................	R
Reaction of truss at B (downward under conditions taken)	R^1

The coordinate of x or x_1 will be measured from A as an origin positive toward C, the center of the span. In general, bending moments will be represented by M and shears by S.

In this case all the moving load is carried to the cable through the suspenders; hence the two reactions R and R^1 will be equal and opposite in direction.

The equations of condition for equilibrium are:

$$R+pl-wx_1+R^1=0 \qquad (1)$$
$$\tfrac{1}{2} \text{ of } pl^2+R^1l-\tfrac{1}{2}wx_1^2=0 \qquad (2)$$
$$R+R^1=0 \qquad (3)$$

Equation (1) at once gives:

$$p=\frac{wx_1}{l} \qquad (4)$$

Then from equations (2) and (3):

$$R=-R^1=\frac{wx_1}{2}\left(1-\frac{x_1}{l}\right) \qquad (5)$$

The bending moment at any section distant x (never greater than x_1) from A is:

$$M=Rx-\left(w-p\right)\frac{x^2}{2}=\frac{wx}{2}\left(x_1-x\right)\left(1-\frac{x_1}{l}\right) \qquad (6)$$

Hence there is always a point of contra-flexure at the head of the moving load.
M has its maximum value for $x=\frac{x_1}{2}$.
Hence:

$$M_1=\frac{wx_1^2}{8}\left(1-\frac{x_1}{l}\right) \qquad (7)$$

M_1 has its maximum value for $x_1=\frac{2}{3}l$.
Hence:

$$M_{max}=\frac{4}{27}\,\frac{wl^2}{8} \qquad (8)$$

The unloaded part of the span acts as a beam simply supported at its ends and carrying an upward load of uniform intensity $p=\frac{wx_1}{l}$. Hence its greatest bending moment at center will be:

$$-M_1=\frac{wx_1}{8l}\left(l-x_1\right)^2 \qquad (9)$$

This has its maximum for $x_1=\frac{2}{3}l$.

Hence:

$$-M_{max}=\frac{4}{27}\,\frac{wl^2}{8} \qquad (10)$$

If any given length of load, as wx_1, move over the span, there will be a point of contra-flexure at each extremity of it, and the greatest bending moment it produces (i. e., at its center) will be given in general by equation (7), or by equation (8) if its length is $\frac{2}{3}l$. The range of the maximum downward bending moment given by equation (8) is the middle third of the span, while the same maximum, as an upward moment, equation (10), occurs at the one-third points only.

The shear at any section distant x (less than x_1) from A (figure 1), is:

$$S=R+px-wx=w\left(1-\frac{x_1}{l}\right)\left(\frac{x_1}{2}-x\right) \qquad (11)$$

When $x=0$ (i. e., at the end of the span) the shear is equal to the reaction R. The greatest shear is found by making $x=0$ and $x_1=\frac{l}{2}$ in equation (11): — $S_{max}=\frac{wl}{8}$. When x is greater than $\frac{x_1}{2}$, the shear S becomes negative and takes its maximum at the head of the load, or for $x=x_1$:

$$S=-\frac{wx_1}{2}\left(1-\frac{x_1}{l}\right)=R^1 \qquad (12)$$

Equations (11) and (12) give the shear when the load wx_1 occupies any position on the span, provided x and x_1 are measured from either end of that loading.

The shear in the unloaded portion of the span $(l-x_1)$ is:

$$S=-R^1-p\,(l-x)=\frac{wx_1}{2}\left\{\left(1-\frac{x_1}{l}\right)-2\left(1-\frac{x}{l}\right)\right. \qquad (13)$$

This expression attains its maximum values for $x=l$ and $x=x_1$, as it then becomes equal to R or R^1; it also becomes zero for $x=\frac{1}{2}(l+x_1)$.

THEORY OF THE CENTER HINGED STIFFENING TRUSS.

There will be given in this appendix only such a concise statement of the theory of the center-hinged stiffening truss as the purposes of this report require, but within those limits it will be complete. Those purposes are satisfactorily served by considering the question as a problem in statics, in which the effects resulting from the deformation of the cable, the elastic stretching of the suspenders, and the elastic deflection of the truss, all under the moving load, are ignored. A complete analysis of the influences of these separate assumptions exerted upon the results obtained would be out of place here, although the effect of the stretching of the suspenders is illustrated at the end of this appendix, but it may be stated that the resultant effect is to confer upon the cable an increased facility of adjustment in form to the requirements of a varying moving load, and hence, an enhanced capacity to receive such moving load directly through the suspenders without action of the truss. The variations from the exact treatment which involves the consideration of elastic deformation of the members named results in formulæ which, as the mathematical theory of the combined elastic action of cables and beams show, give greater bending moments and shears in the stiffening truss over the greater part of the span than those which actually exist. All estimates of material for the members affected will consequently be on the safe side. The influence of the elevation or depression of the cable as a whole, due either to a variation in temperature or to elastic extension, is eliminated by the center hinge.

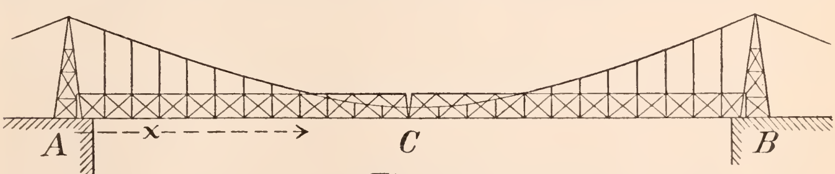


Fig. 2.

The moving load will be taken on the left half truss A C, and the same notation as that given below fig. 1 in the preceding section will be used.

CASE I.

In this case the moving load w will be taken as passing on the truss from A and its variable length measured from that point will be represented by x_1.

Inasmuch as the pull exerted on the truss by the suspenders is upward and as there is no moving load on B C, it is clear that the reaction R^1 at B must be downward in direction and that an equal downward reaction must be exerted by the half truss A C on B C at C, the hinge point at the center of the span. That downward reaction at C is a part of the requisite portion of the moving load $w\,x_1$ determined by the simple principle of the lever applied to A C$=\frac{l}{2}$ as a span, while the other part of

that portion is carried directly to A without exerting any pull on the suspenders, and, consequently, without stressing the cable. The part transferred to C (equal to the downward reaction at B) is evidently less than that transferred to A, except in the case of the moving load covering the half span, when they are equal. From this reasoning it follows that the total upward pull on the suspenders is less than the moving load $w\,x_1$ by the excess of the part transferred to C, but both these differences disappear when the moving load covers the half span.

The forces acting on the entire truss A B are, then, the upward reaction R (at A), the downward reaction R^1 (at B), the upward pull pl, and the downward-moving load $w\,x_1$, and it is necessary for equilibrium that their sum shall be zero.

Hence:

$$pl + R - wx_1 - R^1 = 0 \quad (14)$$

Because the truss is hinged at the point C, moments of the same forces acting on each half, and about C as a center, must be equal to zero.

Hence:

$$\left.\begin{aligned} \frac{pl^2}{8} + \frac{Rl^2}{2} - w\,x_1 \left(\frac{l - x_1}{2}\right) &= 0 \\ \frac{pl^2}{8} - \frac{R^1 l}{2} &= 0 \end{aligned}\right\} \quad (15)$$

By placing the two equations (15) equal to each other

$$R = w\,x_1 - \frac{wx_1^2}{l} - R^1 \quad (16)$$

The substitution of this value of R in equation (14) will give:

$$\frac{pl^2}{8} - \frac{R^1 l}{4} = \frac{wx_1^2}{8} \quad (17)$$

By the combination of the second of equations (15) and equation (17):

$$R^1 = \frac{w\,x_1^2}{2l} \quad (18)$$

By placing this value of R^1 in equation (16):

$$R = wx_1\left(1 - \frac{3x_1}{2l}\right) \quad (19)$$

The value of R^1 from equation (18) placed in the second of equations (15) will yield:

$$p = \frac{2wx_1^2}{l^2} \quad (20)$$

These values of R^1, R, and p, in which x_1 must never exceed $\frac{1}{2}\,l$, will enable all moments and shears to be immediately written.

The expression for the bending moment at any point x (not greater than x_1) from the point of support A is:

$$M = R\,x + p\frac{x^2}{2} - \frac{wx^2}{2} = w\left\{x_1\left(1 - \frac{3x_1}{2l}\right)x + \left(\frac{2x_1^2}{l^2} - 1\right)\frac{x^2}{2}\right\} \quad (21)$$

In order to determine what position of loading must be taken so that the maximum bending moment will exist at any desired section distant x from the point of support A, the first derivative of M in respect to x_1 (x being considered constant) must be put equal to zero and then solved for x_1:

$$\frac{d\,M}{dx_1} = w\left\{x - \frac{3x_1x}{2l} - \frac{3x_1x}{2l} + \frac{2x_1x^2}{l^2}\right\} = 0$$

$$\therefore\ x_1 = \frac{l^2}{3l - 2x} \quad (22)$$

By placing the value of x_1 in equation (21):

$$M_x = \frac{wx}{2}\left\{\frac{l^2}{3l-2x} - x\right\} = \frac{wx}{2}\left\{\frac{l}{3-\frac{2x}{l}} - x\right\} \quad \text{(23)}$$

Equation (23) will give the greatest possible bending moment in the half truss A C at any point indicated or located by the coordinate x which will never exceed $\frac{l}{2}$. The corresponding length of loading from A will be given by x_1 in equation (22).

The greatest bending moment in the half truss A C can be found by placing $\frac{dM_x}{dx} = 0$, then solving this equation for x and placing its value in equation (23); but there is a simple procedure.

Let x_1 be considered constant in equation (21), then if $\frac{dM}{dx} = 0$ there will be located the section of the greatest bending for any given length of loading x_1:

$$\frac{dM}{dx} = w\left\{x_1\left(1-\frac{3x_1}{2l}\right) + \left(\frac{2x_1^2}{l^2} - 1\right)x\right\} = 0$$

$$\therefore x = x_1 \frac{\left(\frac{3x_1}{2l} - 1\right)}{\frac{2x_1^2}{l^2} - 1} \quad \text{(24)}$$

If this value of x be placed in equation (22) there will result:

$$\left(\frac{x_1}{l}\right)^3 - \left(\frac{x_1}{l}\right) + \frac{1}{3} = 0 \quad \text{(25)}$$

Equation (25) is satisfied, for:

$$\frac{x_1}{l} = .395 \quad \text{(26)}$$

Or, the greatest bending moment in the span occurs when 0.395 of its length is covered with the moving load.

If $\frac{x_1}{l}$ is taken from equation (26) and placed in equation (24):

$$x = .234\,l \quad \text{(27)}$$

Equation (27) thus locates the section of greatest bending in the span.
By placing $x = .234\,l$ in equation (23), that maximum moment becomes:

$$M_{max} = .018825 - wl^2 = .1506\,\frac{wl^2}{8} \quad \text{(28)}$$

The half truss B C is simply supported at each end and carries the uniform upward load p per linear foot; hence it will be bent upward with the negative moment:

$$M' = -p\frac{x}{2}\left(\frac{1}{2}l - x\right).$$

This expression evidently has its maximum value when $x_1 = \frac{l}{2}$.

Hence:

$$M'_{max} = -\frac{wx}{4}\left(\frac{1}{2}l - x\right) \quad \text{(29)}$$

In these expressions neither x_1 nor x can ever exceed $\frac{l}{2}$.

With the value $x=\frac{1}{4}l$:

$$M^1_{\max}=-\frac{1}{8}\frac{wl^2}{8}=-\frac{w_\circ}{64} \quad \text{.......... (30)}$$

The shear at any section located by the coordinate x (never exceeding x_1) is:

$$S=px+R-wx=\frac{2w\,x\,x_1^2}{l^2}+wx_1\left(1-\frac{3x_1}{2l}\right)-wx.$$

$$\therefore S=wx\left(\frac{2x_1^2}{l^2}-1\right)+wx_1\left(1-\frac{3x_1}{2l}\right) \quad \text{.......... (31)}$$

In order to determine the greatest shear at any point or section x the same procedure employed for the moment must be followed:

$$\frac{d\,S}{d\,x_1}=\frac{4\,w\,x\,x_1}{l^2}+w-w\frac{3\,x_1}{l}=0 \therefore x_1=\frac{l^2}{3\,l-4x} \quad \text{.......... (32)}$$

This value of x_1 placed in equation (31) gives:

$$S_1=w\left\{\frac{l^2}{2\,(3l-4x}-x\right\} \quad \text{.......... (33)}$$

The value S_1 in equation (33) gives the greatest shear in the half truss at any section x.

The form of equation (33) shows that the maximum of all the values of S_1 will be found by making $x=0$; hence:

$$S_{\max}=\frac{wl}{6} \quad \text{.......... (34)}$$

By making $x=0$ in equation (32) it appears that the length of moving load required to produce the maximum shear is:

$$x_1=\frac{l}{3} \quad \text{.......... (35)}$$

This value of x_1 placed in equation (19) shows that the maximum shear is equal to the maximum reaction R.

The preceding equations have been so written that an upward shear is positive and a downward shear negative. If S_1 be placed equal to zero, by using equation (33):

$$x=\frac{l}{4} \text{ or } \frac{l}{2} \quad \text{.......... (36)}$$

The second value indicates nothing useful, but the first value $\left(\frac{l}{4}\right)$ shows that the shear is always upward over that quarter of the span nearest A, and downward or negative over that quarter adjacent to C. The greatest negative shears will obviously occur at the head of the moving load, and they will be given by making $x=x_1$ in equation (31);

$$-S_1=\frac{wx_1^2}{l}\left(\frac{2x_1}{l}-\frac{3}{2}\right) \quad \text{.......... (37)}$$

For a maximum:

$$x_1=\frac{l}{2} \text{ and } -S_1=-\frac{wl}{8}$$

The preceding reasoning shows that *the maximum negative shears will be given by equation (37) in which x_1 must have values between $\frac{l}{4}$ and $\frac{l}{2}$ only;* but that *the maximum positive shears will be given by equation (33), in which x must have values between $x=0$ and $x=\frac{l}{4}$ only.*

Since for $x_1 = \frac{l}{2}$, $R^1 = \frac{wl}{8}$ and $p = \frac{w}{2}$, the expression for the shear in the half truss B C (or in the half truss A C fully loaded) will be:

$$S^1 = w\left(\frac{l}{8} - \frac{x}{2}\right) \qquad (38)$$

By increasing x from 0 to $\frac{l}{2}$ in equation (38) there will result the positive shears in the right half of B C and the negative shears in the left half of B C.

CASE II.

In this case a moving load of length varying from 0 to $\frac{l}{2}$ will be supposed to be placed on the span and to extend from the center C to any point distant $\left(\frac{1}{2}\,l - x_1\right)$ from C. All notation will remain as in Case I.

The equations of condition for equilibrium corresponding to equations (14) and (15) now become:

$$pl + R - w\left(\frac{l}{2} - x_1\right) - R^1 = 0 \qquad (39)$$

$$\left.\begin{aligned} \frac{pl^2}{8} + \frac{Rl}{2} - \frac{w}{2}\left(\frac{l}{2} - x_1\right)^2 &= 0 \\ \frac{pl^2}{8} - \frac{R^1 l}{2} &= 0 \end{aligned}\right\} \qquad (40)$$

The two equations (40) give:

$$R = \frac{w}{l}\left(\frac{l}{2} - x_1\right)^2 - R^1 \qquad (41)$$

The substitutions of R from equation (41) and pl from the second of equations (40) in equation (24) give:

$$R^1 = \frac{w}{2}\left(\frac{l}{2} - x_1\right)\left(\frac{x_1}{l} + \frac{1}{2}\right) \qquad (42)$$

By placing this value of R^1 in equation (41):

$$R = \frac{w}{2}\left(\frac{l}{2} - x_1\right)\left(\frac{1}{2} - \frac{3x_1}{l}\right) \qquad (43)$$

The same value of R^1 in the second of equations (40):

$$p = \frac{2w}{l}\left(\frac{l}{2} - x_1\right)\left(\frac{x_1}{l} + \frac{1}{2}\right) \qquad (44)$$

The reactions R and R^1 will evidently have their greatest values when $x_1 = 0$:

$$R_{\max} = R^1_{\max} = \frac{wl}{8} \qquad (45)$$

The greatest values of two bending moments must be determined—i. e., one at that extremity of the moving load at the distance x_1, from A, fig. 2, and the other at any point of A C with that half span entirely covered with the moving load. The expression for the moment at any point x between the end A and x_1 is:

$$M = Rx + \frac{1}{2}\,px^2 = w\left(\frac{l}{2} - x_1\right)\left\{\frac{x}{2}\left(\frac{1}{2} - \frac{3x_1}{l}\right) + \frac{x^2}{l}\left(\frac{x_1}{l} + \frac{1}{2}\right)\right\} \qquad (46)$$

The form of this expression shows it to be a maximum for $x = x_1$:

$$M_1 = wx_1\,l\left(\frac{1}{2} - \frac{x_1}{l}\right)^3 \qquad (47)$$

In order to find a maximum:

$$\frac{dM_1}{dx_1}=w\left\{l\left(\frac{1}{2}-\frac{x_1}{l}\right)^3-3x_1\left(\frac{1}{2}-\frac{x_1}{l}\right)^2\right\}=0$$

Hence $x_1=\frac{l}{8}$;

Which placed in equation (47):

$$M_{1\,max}=\frac{wl^2}{8}\cdot\frac{27}{512}=.0527\,\frac{wl^2}{8} \quad\text{(48)}$$

If the moving load covers the whole of the half span A C, the moment at any point x will be, since then $p=\frac{w}{2}$:

$$M^1=(w-p)\frac{x}{2}\left(\frac{1}{2}l-x\right)=\frac{wx}{4}\left(\frac{1}{2}l-x\right) \quad\text{(49)}$$

This is the maximum for $x=\frac{l}{4}$:

$$M^1_{max}=.125\,\frac{wl^2}{8}=\frac{wl^2}{64} \quad\text{(50)}$$

The bending in the right half of the span is given by equations (29) and (30) in the preceding case.

The general expression for the upward (positive) shear in the unloaded portion x_1 of the left half, A C, of the span is:

$$S=R+px=w\left(\frac{l}{2}-x_1\right)\left(\frac{1}{4}-\frac{3x_1}{2l}+\frac{2xx_1}{l^2}+\frac{x}{l}\right) \quad\text{(51)}$$

For the greatest value of S, always $x=x_1$:

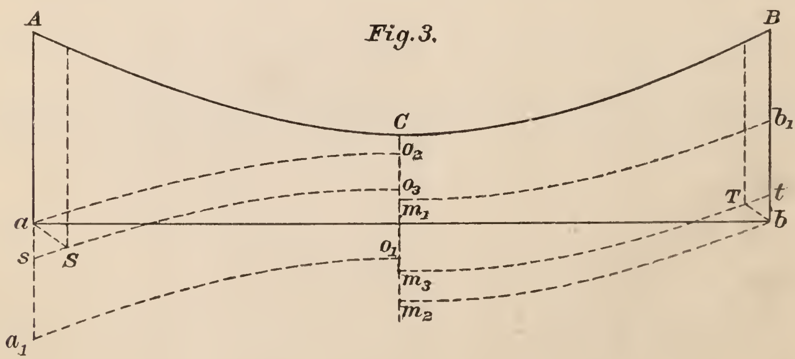
$$S_1=w\left(\frac{l}{2}-x_1\right)\left\{\frac{1}{2}\left(\frac{1}{2}-\frac{x_1}{l}\right)+\frac{2x_1^2}{l^2}\right\} \quad\text{(52)}$$

The shears in each half of the span with the left half, A C, fully loaded, have been given in the preceding case by equation (38).

EFFECT OF CHANGES OF LENGTHS OF THE SUSPENDERS ON THE DISTRIBUTIVE FUNCTION OF THE STIFFENING GIRDER.

The suspenders are lengthened or shortened by changes of temperature; they are lengthened by the live load; their increment of length due to either cause are pro-

Fig. 3.



portional to their length; these increments are, therefore, ordinates of parabolas.

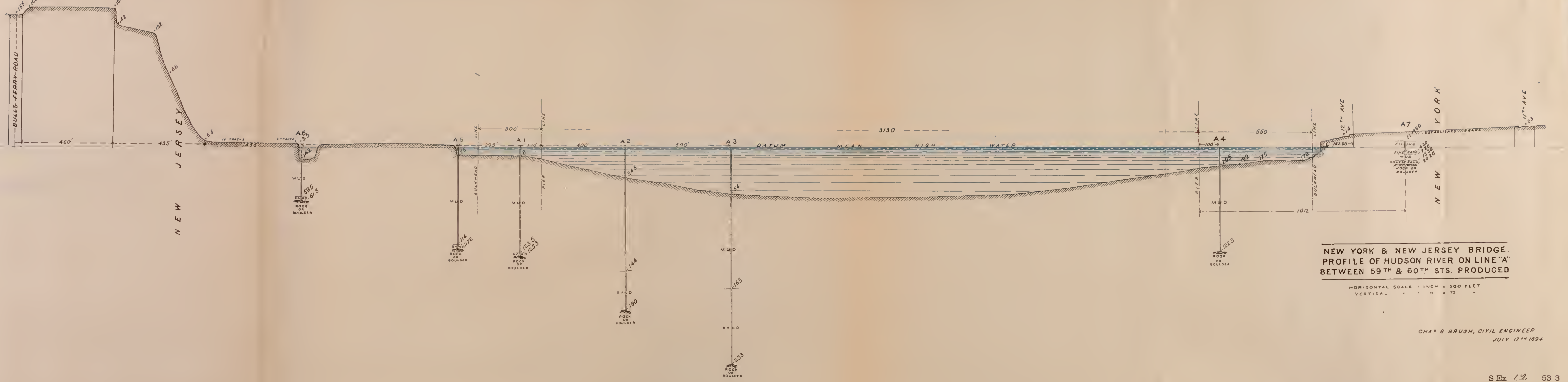
Referring to fig. 3, where A C B represents the cable, *a c b* the stiffening girder, and A *a*, B *b*, the towers, if the increments of length of the suspenders, due to a rise of tem-

perature, be plotted downward from line $a\ b$, they will give the parabolic arc $a_1\ o_1$; supposing that the towers expand in the same proportion as the suspenders, the curve $a_1\ o_1$ becomes $a\ o_2$ and represents, in position, the deflected line of the girder, this line is similar to the curve of deflection due to a uniform rate of loading, and has a slight convexity upward, showing that no disturbance has taken place in the distributive action of the girder, except a slight relief of duty for the loaded part, and a slight increase of duty for the unloaded part, of the girder. Owing, however, to the greater bulk of material in the tower, its temperature and rate of expansion will be less than for the suspenders; if $a\ s$ represents the difference due to this cause, then the curve, $a\ o_2$ drops to the position $s\ o_3$, and a S o_3 becomes the line of deflection of the girder, showing that in the immediate vicinity of the towers the suspenders are partly relieved of their duty, and that a part of the load is carried directly to the tower by the girder.

A similar construction for a fall of temperature, to the right of the figure, shows the effect on the girder to be slight deflection downward with the convexity of the curve downward, indicating a slight increase of duty for the loaded part of the girder and a slight decrease for the unloaded part, owing to the difference of contraction $b\ t$ of the tower, the deflected line of the girder will be b T m_3, showing that the duty of the suspenders in the immediate vicinity of the tower has been increased.

The effect of the elastic elongation of the suspenders due to the live load is exactly the same as it is for a rise of temperature, except that owing to the fact that the tower contracts instead of expanding, the disturbing effects in the vicinity of the tower is increased in warm weather and decreased in cold weather.

The ill effects on the girders and suspenders arising from these disturbances are avoided by omitting the suspenders for a short distance next to the towers.



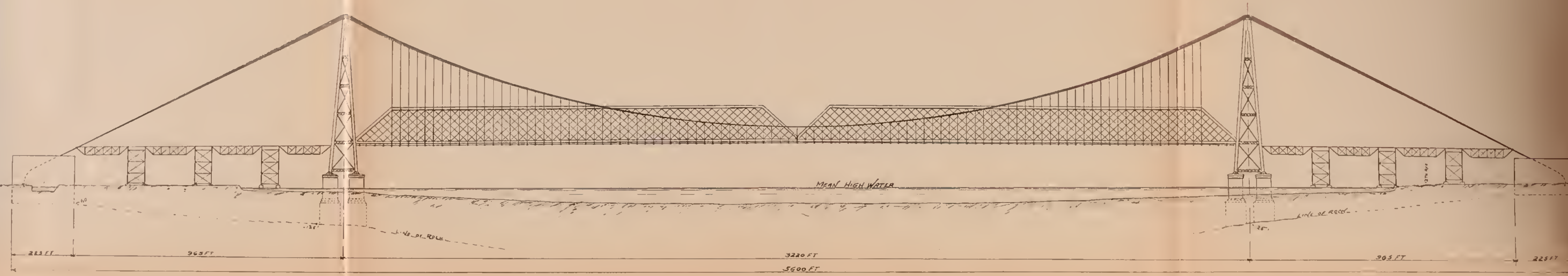
NEW YORK & NEW JERSEY BRIDGE.
PROFILE OF HUDSON RIVER ON LINE "A"
BETWEEN 59TH & 60TH STS. PRODUCED

HORIZONTAL SCALE 1 INCH = 300 FEET.
VERTICAL " 1 " = 75 "

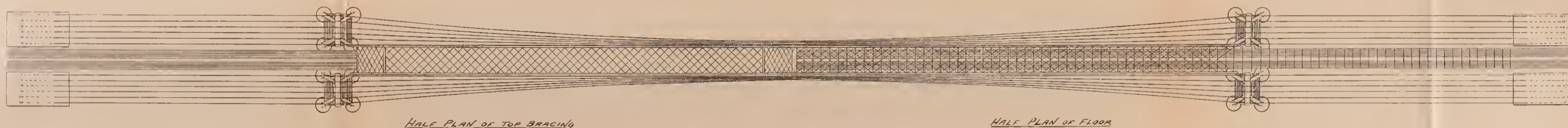
CHAS B. BRUSH, CIVIL ENGINEER
JULY 17TH 1894

S Ex 12. 53 3


Pier head
Rock.
Pier head
Rock.
1575
3310'
885
4310'



SIDE VIEW OF BRIDGE.

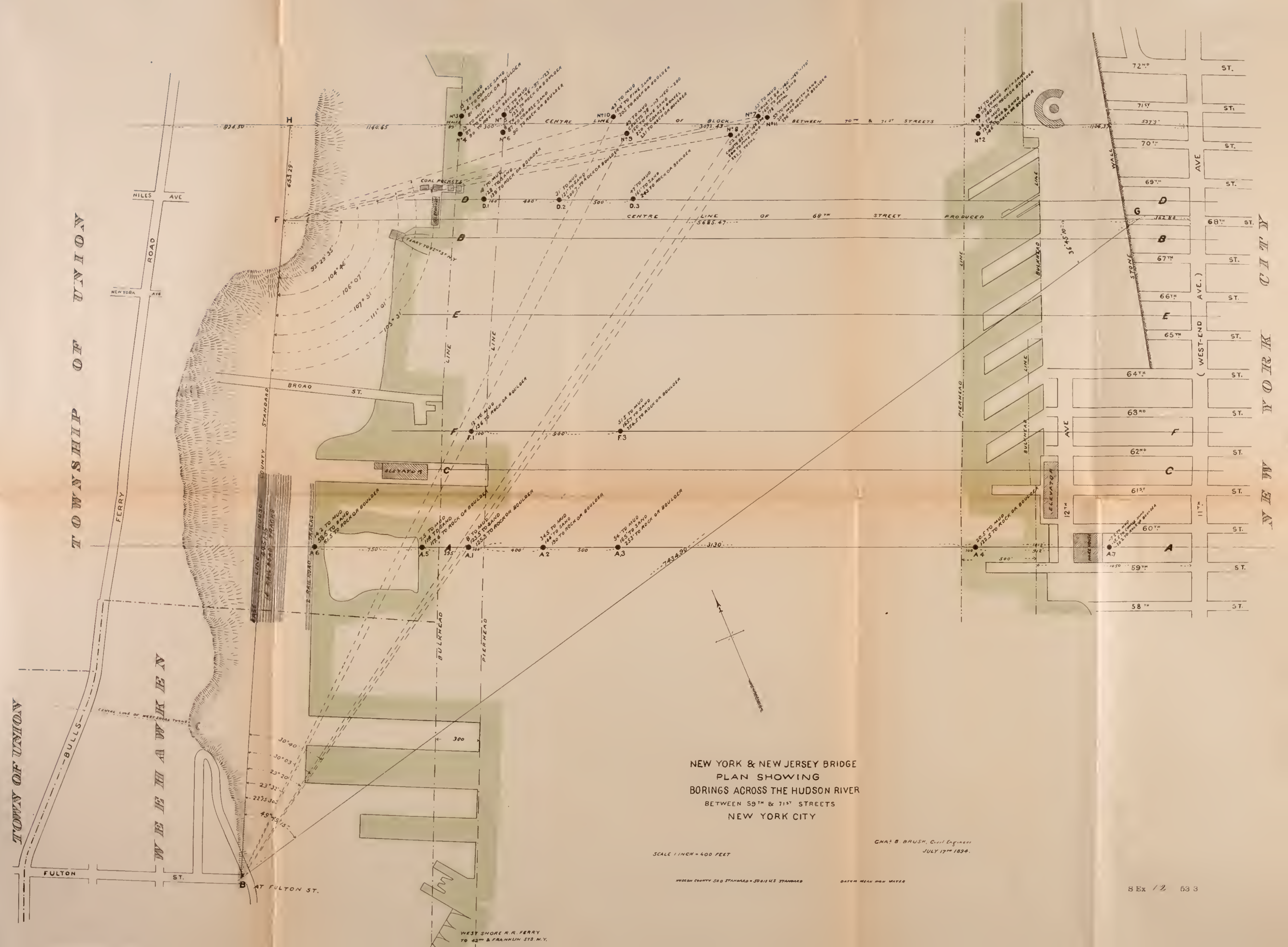


PLAN OF BRIDGE.

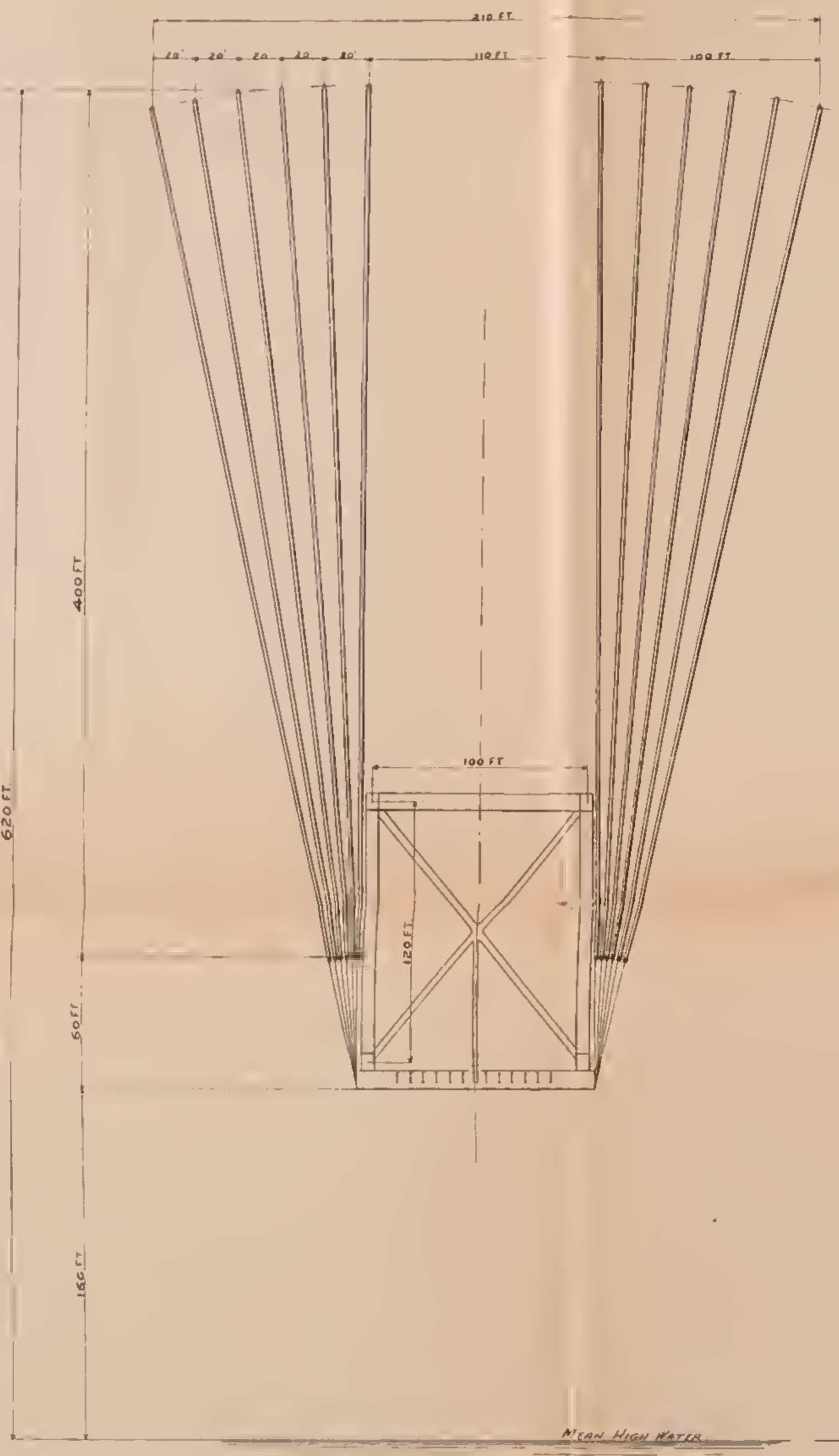
PROPOSED HUDSON RIVER SUSPENSION BRIDGE

TO ACCOMPANY REPORT OF BOARD OF BRIDGE ENGINEERS
APPOINTED UNDER ACT OF CONGRESS APPROVED JUNE 7-1894.

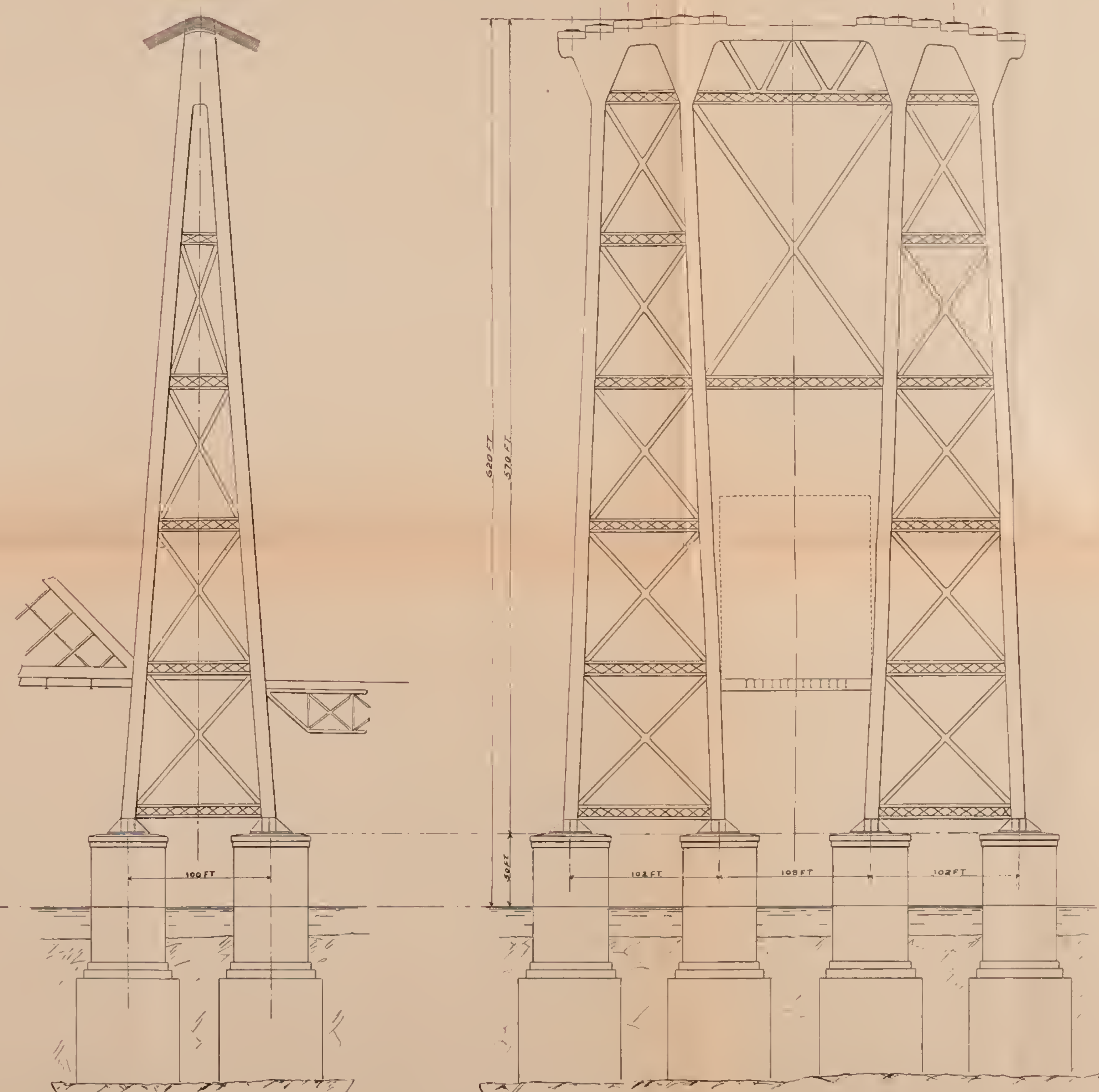
SCALE: 300 FT. = 1 IN.



NEW YORK & NEW JERSEY BRIDGE
PLAN SHOWING
BORINGS ACROSS THE HUDSON RIVER
BETWEEN 59TH & 71ST STREETS
NEW YORK CITY
CHAS. B. BRUSH, Civil Engineer
JULY 17TH 1894.
SCALE 1 INCH = 400 FEET
DATUM MEAN HIGH WATER
TOWNSHIP OF UNION
TOWN OF UNION
WEEHAWKEN
NEW YORK CITY
NILES AVE
NEW YORK AVE
ROAD
FERRY
BULLS
FULTON ST.
AT FULTON ST.
BROAD ST.
HUDSON COUNTY STANDARD
BASE LINE
CENTRE LINE OF BLOCK BETWEEN 70TH & 71ST STREETS
CENTRE LINE OF 68TH STREET PRODUCED
COAL POCKETS
FERRY TO 42ND ST N.Y.
ELEVATOR
BULKHEAD
PIERHEAD
LINE
WEST SHORE R.R. FERRY TO 42ND & FRANKLIN STS. N.Y.
72ND
71ST
70TH
69TH
68TH
67TH
66TH
65TH
64TH
63RD
62ND
61ST
60TH
59TH
58TH
ST.
AVE
(WEST-END AVE.)
11TH
12TH AVE
STONE WALL
7434.90
3130
5685.47



SECTION THROUGH BRIDGE AT CENTER



ELEVATIONS OF TOWER AND FOUNDATION.

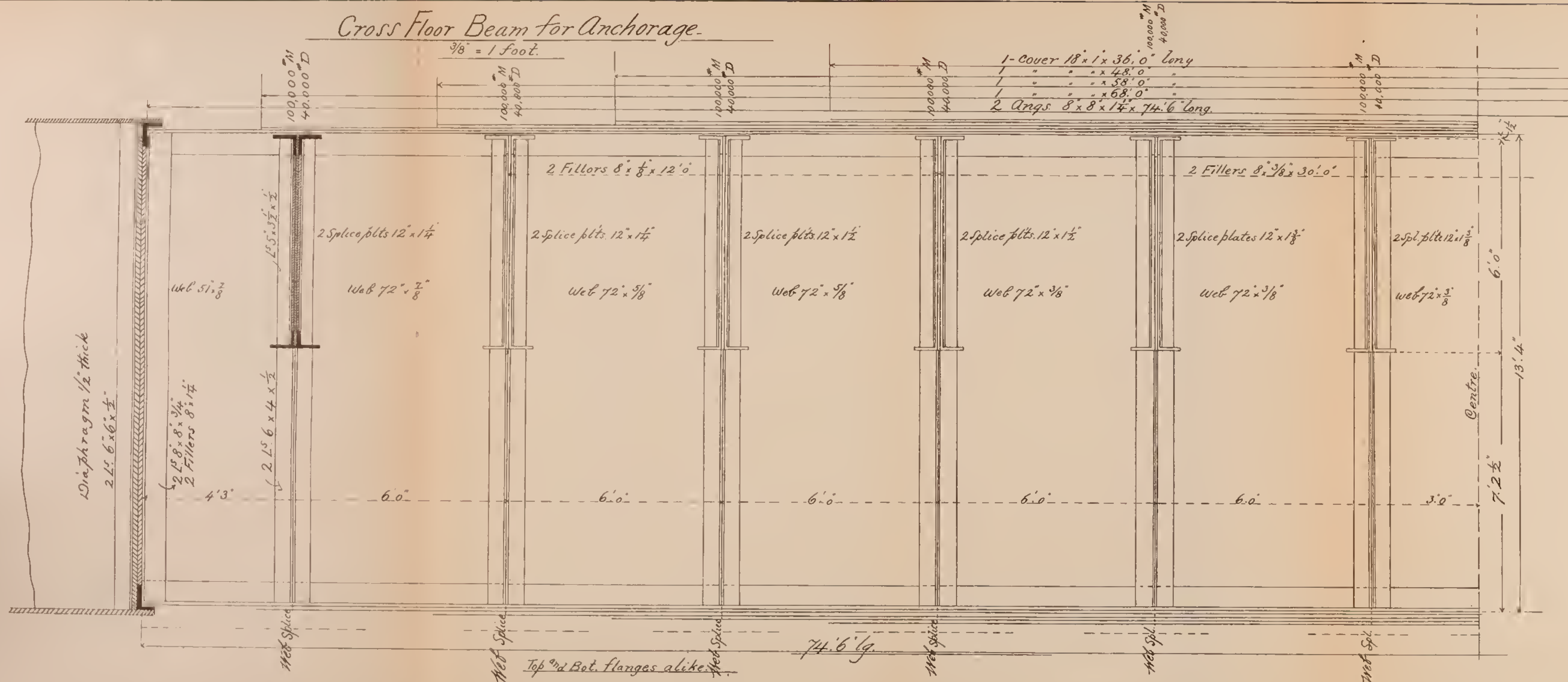
PROPOSED HUDSON RIVER SUSPENSION BRIDGE

TO ACCOMPANY REPORT OF BOARD OF BRIDGE ENGINEERS
APPOINTED UNDER ACT OF CONGRESS APPROVED JUNE 7-1894.

SCALE: 75 FT. = 1 IN.

Cross Floor Beam for Anchorage

3/8" = 1 foot.



Union Bridge Company

Allowed Strains.

In Stringers

Tension on bot. flanges 10.000# per □ net
Top flange same as bottom

In Cross Floor Beams.

Tension on bot. flange 13 600# per □ net.
Top flange same as bottom.

On Trusses.

$a = 12.000\left(1 + \frac{\text{minimum}}{\text{maximum}}\right)$ For tension.

$a = \left(12.000 - 40\frac{l}{r}\right)\left(1 + \frac{\text{minimum}}{\text{maximum}}\right)$ For Comp. on Chds. & Towers.

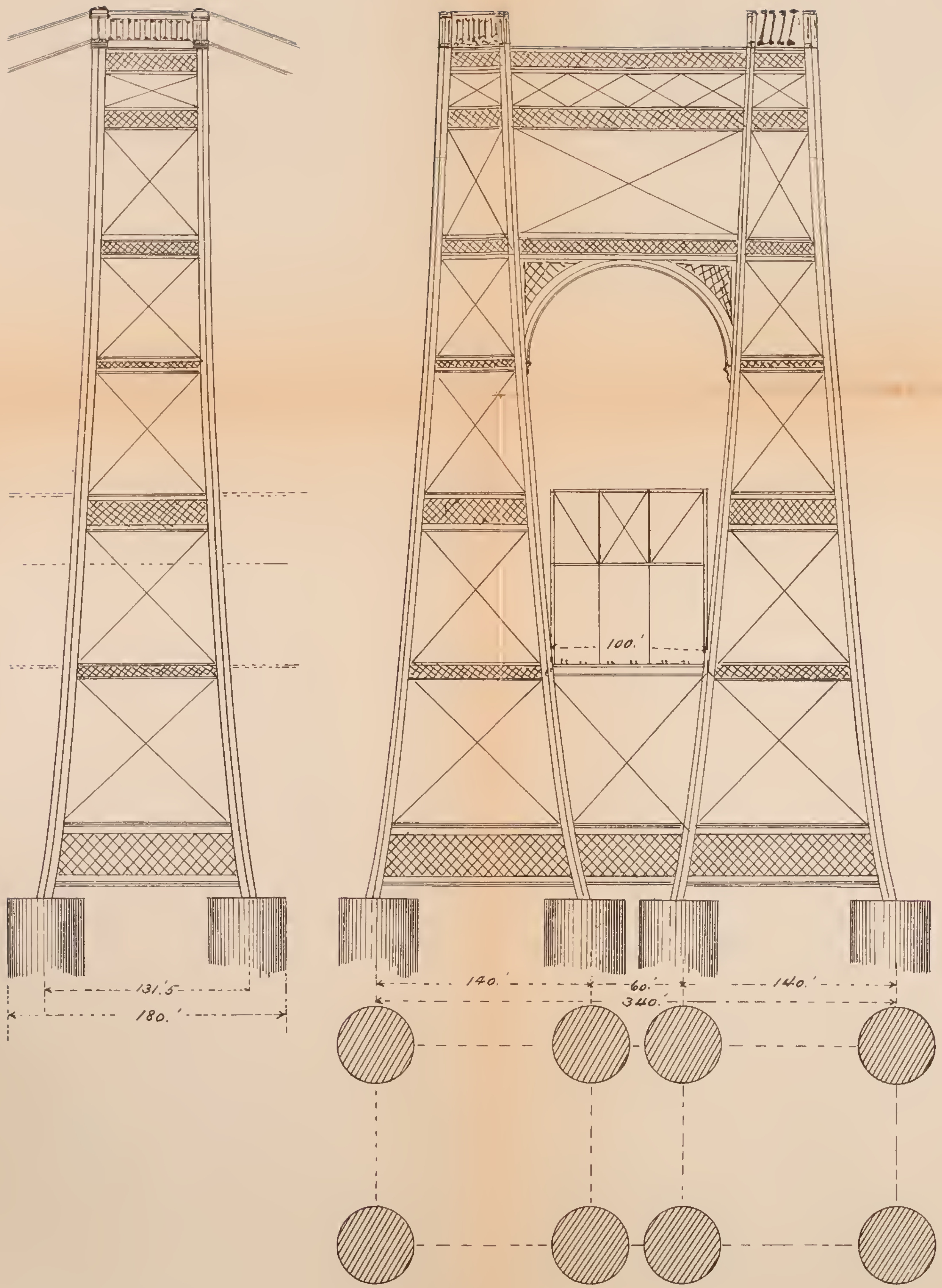
$a = \left(12.000 - 60\frac{l}{r}\right)\left(1 + \frac{\text{minimum}}{\text{maximum}}\right)$ For Comp. on Posts.

For members subject to Tension & Comp.

For Tension $= 12.000\left(1 - \frac{4}{10}\frac{\text{Comp.}}{\text{Tension}}\right)$

For Comp. $= \left(12.000 - 40\frac{l}{r}\right)\left(1 - \frac{4}{10}\frac{\text{Tension}}{\text{Comp}}\right)$ Chords

For Comp $= \left(12.000 - 60\frac{l}{r}\right)\left(1 - \frac{4}{10}\frac{\text{Tension}}{\text{Comp.}}\right)$ Posts.



Sketch of Towers.
Scale 1/8" = 10'.

Bridge over the Hudson River between New York & Jersey City.
Scale 1" = 60 feet.
M signifies Moving Loads.
D " Dead "
e " Erection "
Figures at panel points indicate concentrated dead loads.
W signifies wind strain.
Moving load per linial foot of bridge 18.000# for 6 tracks
Union Bridge Co.
Dead load for Suspended Span 24.000# per linial foot.
14 panels of 60 ft = 840 feet
12 panels of 60 ft = 720 feet
6 panels of 60 ft = 360 feet
200 feet
High Water
Stringers for 60 feet panels.
Rolling load on each Stringer 2,000# per ft.
1-Cover 13 x 1/2 x 35'0"
1 " 13 x 1/2 x 50'0"
2-Angs 6 x 4 x 1/2 x 60'0"
Top flange.
1-Web 72 x 3/8 x 60'0"
2-Angs 6 x 4 x 1/2 x 60'0"
1-Cover 13 x 1/2 x 35'0"
1 " 13 x 1/2 x 50'0"
Bot flange.
12 Stringers for each panel.
Centre
Top Laterals.
Bot Laterals.